LONG TERM DEFORMATIONS
OF CONCRETE MADE WITH
FLORIDA AGGREGATES

By

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LONG TERM DEFORMATIONS OF CONCRETE MADE WITH FLORIDA AGGREGATES

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The long term deformation of concretes containing Florida aggregates has become important with the recent introduction of long span precast segmental bridges in Florida. To improve the understanding of the creep and shrinkage behavior of these concretes a study was conducted and the results were compared with those obtained by accepted prediction methods.

Concrete mixtures prepared with Florida aggregates known as Brooksville, Miami Oolite, and Ft. Thompson were tested over a period of one year to evaluate creep and shrinkage. During the same period concrete prepared with another aggregate, Calera, from Alabama, was tested in the same manner to serve as a reference. The Calera aggregate is a nonporous aggregate as compared to the Florida aggregates which are very porous. The study evaluated the effects of age of loading, relative humidity of the storage environment, the level of applied stress, and aggregate source on creep and shrinkage. These results were compared with those obtained from three methods of predicting creep and shrinkage; the ACI model, the CEB model and the BP model. In addition, the creep data from the study were fitted to a Burgers model for study of their rheological properties.

This study did not find any substantial difference in the creep and shrinkage properties of concrete containing Florida aggregates as compared with concrete containing the nonporous Calera aggregate. The relative humidity of the storage environment had the largest effect on creep. The age of loading was only significant at very early ages of the concrete and the level of stress had no substantial effect.

The three creep prediction models did not correspond well to actual creep measurements. The ACI model did acceptably predict the shrinkage but the other models did not.

CHAPTER I
INTRODUCTION

Nature of the Problem

The recent trend in the construction of highway bridges is toward the use of concrete box girders and longer span lengths. With these longer spans has come an increased awareness that the creep characteristics of concrete can have a significant impact on the structure. The effect of the coarse aggregate used in making the concrete has been known to affect the creep characteristics of the concrete.

Creep data for concrete containing Florida aggregates are not available. Generally, weakly cemented and more porous aggregates yield higher concrete creep rates. Even high quality aggregates in Florida tend to have a water absorption capability that is greater than most aggregate found in other states. This implies that the aggregates are more porous and may have increased compressibility and creep characteristics. Florida aggregates have been used satisfactorily in concrete structures and there is no indication of poor performance except in isolated areas.

Creep and shrinkage may or may not be desirable depending upon the situation. For cast-in-place structures, creep of relatively low strength fresh concrete adds ductility and hence relief at highly stressed locations within a newly built structure. However, creep results in some loss of effectiveness of steel in prestressed concrete members and could lead to undesirable deformations.

Because of the difficulty in measuring small strains over a long time and the need to measure concrete shrinkage, creep testing is not a simple process. Many factors such as stress level, age at loading, temperature, and ambient humidity influence the creep rate and thus must be controlled or accounted for in some manner so as to obtain meaningful results. Because of these difficulties creep testing is not usually performed.

Several design codes attempt to define creep deformations but their applicability for many situations is doubtful. In order to determine the applicability of the codes more testing of concrete for its creep characteristics is required.

Objectives and Scope

A detailed review and discussion of pertinent previous investigations of the time dependent behavior of concrete when subjected to a uniform stress level forms the initial portion of this dissertation. The investigations reviewed include both experimental and analytical studies of the time dependent behavior of concrete. The object of this review (Chapter II) is to provide an integrated description of the factors influencing creep of concrete. A discussion of current design practices is also presented.

The primary purpose of this research is to determine the creep characteristics of concrete prepared with Florida aggregates. Specific objectives of the research are

1. To measure elastic, delayed elastic, creep, and shrinkage characteristics of concrete specimens tested under constant compressive stress.
2. To compare creep properties of concrete prepared using three Florida aggregates with concrete containing a relatively nonporous aggregate.
3. To evaluate the potential for using the indirect tensile test for measurement of tensile creep properties and for correlation to compressive creep properties.
4. To evaluate the applicability of current codes and if necessary recommend modifications to improve their reliability.

The experimental study was limited to three Florida aggregates and one out-of-state aggregate which was used as a reference. Various uniform loads were used in combination with different ages at loading and different humidity during storage. Measurement of deformations were made over a period of at least one year of the age of the concrete.

Chapter III describes the materials, specimens and instrumentation used in this study. Chapter IV outlines the test procedure. Test results are presented in Chapter V. In Chapter VI a comparison of prediction models with the data is presented. Chapter VII presents the conclusions reached by this study.

CHAPTER II
LITERATURE REVIEW

Introduction

Concrete has been in use since ancient times but it was only at the beginning of the present century that the property of continuing deformation in concrete when subjected to a sustained loading has been recognized. Shrinkage and swelling of concrete due to drying or wetting, on the other hand, can take place without an external load.

When portland cement concrete was introduced in the 19th century, the design of concrete structures was based on the assumption that concrete behaved elastically. Steel and cast iron structures were designed successfully using the assumption that they behaved elastically within the range of loading of the structure and concrete structures were designed likewise. Steel and other metals do experience creep but only at stresses exceeding the yield stress. Concrete exhibits creep at substantially lower stresses.

Woolson is believed by many researchers to be the first to report, in 1905, the ability of concrete to "flow" under stress. Since then the study of creep has received considerable attention. Neville (39) reported in 1970 that over 1300 papers and reports dealing with the creep of concrete had been published. The studies on creep of concrete generally can be divided into two categories, experimental and theoretical studies and empirical studies. In the first category are the experiments made to understand the cause of creep and the factors that influence creep. Various theories and hypotheses on the mechanism of creep have been advanced, based upon these studies. In the second category the cause is not as important as an attempt to define mathematically the behavior of concrete under the action of sustained loading. Many analytical models, which are generally empirical in nature, have been suggested. Some of the more popular models will be examined in more detail later in this chapter.

This chapter will review the various theories and hypotheses of concrete creep, the general factors that influence creep and shrinkage of concrete and several analytical models of creep and shrinkage.

Definition of Creep

Creep is the deformation produced by a constant sustaining stress (39). Figure 2.1 shows a typical creep curve for concrete. The initial strain on loading depends on the measuring technique used in a particular case such that if the load is applied slowly, some creep will take place and influence the result. After the initial strain the loading strain continues at a decreasing rate with time. This portion is shown in Figure 2.1 and can consist of basic creep

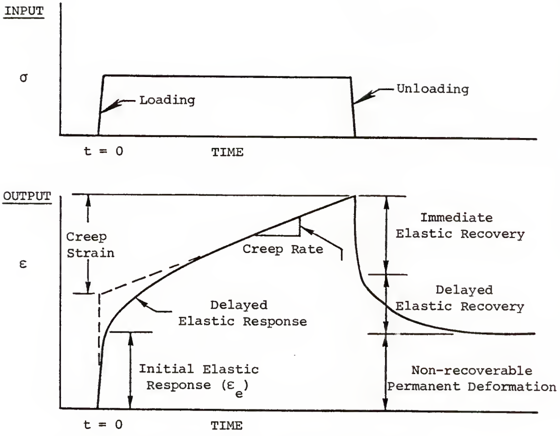


Figure 2.1. Typical Creep Response Curve.

or basic creep and drying creep as defined below. If the specimen is unloaded, instantaneous recovery takes place which is analagous to the instantaneous strain on loading but may not be of the same magnitude. After the instantaneous recovery is the delayed elastic recovery which tends to a finite value. The remaining residual deformation or permanent set can be considered to be the result of creep strain accumulated during loading.

The discussion of creep is made more difficult due to confusion of terminology. This paper will follow the terminology adopted by Neville (39):

Creep--A deformation produced by a constant sustaining stress.

Relaxation--A decrease in stress with time under a constant deformation.

Drying Shrinkage--Shrinkage caused by the withdrawal of water from concrete stored in unsaturated air.

Basic Creep--Creep that occurs when concrete is stored under such conditions that no shrinkage or swelling can take place.

Drying Creep--Creep greater than basic creep that occurs when concrete is stored such that shrinkage takes place.

Strain on Loading--Instantaneous elastic strain at the time of application of the load.

Instantaneous Recovery--The elastic strain recovery that occurs instantaneously after the stress is removed.

Delayed Elastic Recovery--Time dependent recovery that follows the instantaneous recovery.

Residual Strain--The irrecoverable strain or permanent set that exists after the instantaneous and delayed elastic recovery have taken place.

Shrinkage

Shrinkage can be separated into plastic shrinkage which takes place while the concrete is still in the plastic state and that shrinkage which occurs after the concrete has set. Only that shrinkage that occurs after the concrete has set is of interest in the study of creep.

The most important components of shrinkage after the concrete has set is carbonation shrinkage and drying shrinkage. Carbonation shrinkage occurs due to the action of carbon dioxide in the atmosphere reacting, in the presence of water with hydrated cement minerals. Carbonation shrinkage is generally not thought to significantly influence creep because it only affects a small portion, the surface area, of structural concrete, but results of experiments performed by Parrott (94) in the laboratory indicate that carbonation under load can increase the creep by more than 50% on small specimens (12.5 x 12.5 x 110 mm). These results do not indicate a problem with carbonation of concrete structures but in interpreting experimental results derived from small specimens. Carbonation shrinkage is thus probably not relevant to creep in actual structures (15). Throughout the remainder of this paper shrinkage will refer to drying shrinkage.

Drying shrinkage (or wetting swelling) is defined to be the volume change of the concrete as its moisture content is changed. In the case of drying shrinkage a part of this movement is irreversible. The change in volume of drying is not equal to the volume of water removed. Little or no shrinkage takes place from the loss of free water which takes place first. As drying continues, adsorbed water, is removed and a decrease in volume takes place.

The ingredients that make up the concrete are usually the main variables in experimental studies. Many studies have been concerned with the influence of variation in the quantities and properties of these ingredients on the drying shrinkage of concrete. In addition, changes in environmental conditions have been recognized to be of great importance. The effects of these variables on the drying shrinkage are presented below.

Effect of Cement Paste

For a given cement and a given type of aggregate, the amount of shrinkage was found to be approximately proportional to the amount of cement paste in the concrete (34).

Effect of Aggregates

Since aggregate usually occupy about 60 to 80 percent of the total volume of concrete it is expected that they may have important effects on the shrinkage. The aggregate itself normally does not experience any volume change but it does restrain the volume change of the cement paste due to drying. Large differences in shrinkage occur from the use of different aggregates. Troxell et al. (49) states that concretes low in shrinkage generally contain rigid low shrinkage aggregates such as quartz and limestone whereas concrete high in shrinkage contains aggregates that shrink considerably in themselves or have low rigidity. The higher the modulus of elasticity the greater the restraint to shrinkage offered by the aggregates. Lightweight concretes generally shrink more than normal weight concretes. Hamada et al. (31), in a study with a rigid normal weight aggregate, basalt, and with lightweight aggregates of volcanic cinder composition found the normal weight concrete to have a higher shrinkage than the lightweight concretes. The Hawaiian aggregate concretes had nominal compressive strengths of 5,000 psi and were stored at 73 degrees F and 50 percent relative humidity. The basalt concrete had a unit weight of 152 lb/ft^3, whereas the cinder concretes were lighter with unit weights of 124 lb/ft^3 and 121 lb/ft^3.

Both Troxell et al. (49) and Neville (39) report that aggregate gradation has little if any effect on the shrinkage of concrete for a given maximum aggregate size. Larger aggregates result in lower shrinkages because they permit the use of a leaner mix which results in lower shrinkage. These results are confirmed by the findings of Li and Ramakrishnan (32) that concrete made with gap-graded aggregate behaved in a manner similar to that of a continuously graded concrete.

Effect of Chemical Composition and Fineness of Cement

The fineness of cement was once thought to affect the shrinkage but recent studies (20) have not borne out this assumption. Likewise, the chemical composition of cement does not appear to affect shrinkage unless the cement is deficient in gypsum (39).

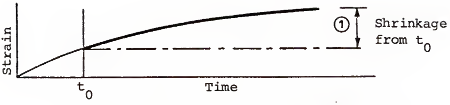
Effect of Admixtures

There are a multitude of admixtures on the market and a thorough study is beyond the scope of this paper so only two admixtures will be examined. The first to be examined is the air entraining admixtures. Troxell et al. (49) report that, as the entrained air is increased, the drying shrinkage is increased. Because concrete with entrained air usually has the same consistency at a lower water cement ratio the shrinkage is not appreciably increased.

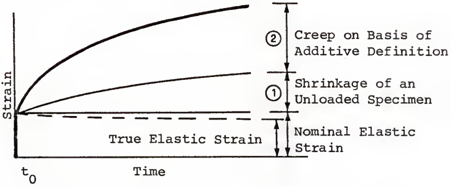
The effect of the second admixture, superplasticizer, on the shrinkage of concrete has received considerable research in the last ten years. Research has not yet given a definitive answer as to whether superplasticizers increase or reduce shrinkage. A number of researchers (22,41) have reported no appreciable difference in shrinkage than base concrete. Several researchers have reported increased shrinkage (23,26,27) and some researchers (21,42,47,48) decreased shrinkage. These differences could be due to the effect of different formulations of superplasticizers used in the research.

Creep

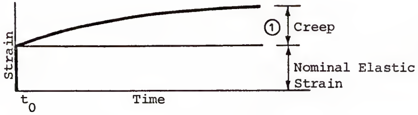
Creep has been defined previously as the increase in strain under a sustaining stress (40). Figure 2.2 shows the relationship between elastic strain, shrinkage, basic creep and drying creep. The nature of creep is still controversial and cannot be discussed here in full. There is no argument that creep is caused by the cement paste and that it is related to internal movement of adsorbed intracrystalline water. Glucklich (30) showed that creep does not take place in concrete in which all the evaporable water has been removed. This situation must be distinguished from older concrete that is in a state of hygral equilibrium but still has evaporable water. Buettner and Hollrah (24) have shown that in this case creep will still take place.



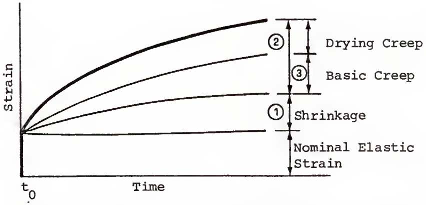
a) Shrinkage of an Unloaded Specimen


Strain
②
Creep on Basis of Additive Definition
①
Shrinkage of an Unloaded Specimen
True Elastic Strain
Nominal Elastic Strain
t_0
Time

b) Change in Strain of Loaded and Drying Specimen



c) Creep of a Loaded Specimen in Hygral Equilibrium with the Ambient Medium.



d) Change in Strain of a Loaded and Drying Specimen.

Figure 2.2. Time-Dependent Deformations in Concrete Subjected to a Sustained Load.

Mechanisms of Creep

There are several general mechanisms of creep that have been proposed. They are: mechanical deformation theory, viscous flow, plastic flow, seepage of gel water, and microcracking. These mechanisms will now be discussed.

Mechanical deformation theory

This theory postulates that when concrete is placed under a compressive stress the capillary spaces are deformed with hygral equilibrium being upset, with the result that water will evaporate until hygral equilibrium is reestablished. The tension in the capillary water rises and to maintain equilibrium the compression in the solid phase increases. According to this theory, no longer accepted, the deformation that results constitutes creep, which is assumed to be completely reversible.

Plastic theories

This theory postulates that creep in concrete is the same as in metals--crystalline flow, i.e. a result of slipping along planes within the crytal lattice. Plasticity does not appear to be a significant cause of creep in concrete. Plastic flow in metals occurs beyond the yield point but in cement the yield point is so low as to be undefined (39).

Viscous and visco-elastic flow theories

In this theory the hydrated cement paste is a highly viscous liquid whose viscosity increases with time as a result of chemical changes within the structure. These changes could be in the form of a coarsening of the particles and thus decrease the specific surface of the cement paste particles. Neville (39) suggests that viscous flow could be the cause of very long term creep.

Seepage theory

According to this theory internal seepage of adsorbed colloidal water due to pressure gradients caused by drying or external load is the cause of shrinkage and creep in concrete. The flow of water to or from the gel takes place through minute capillary channels. The flow along the capillaries depends upon the vapor pressure gradient and the friction along the channels. Creep is a function of the relative amount of the unfilled space. Neville (39) believes that slow long term creep is due to other causes than seepage but can only happen if some evaporable water is present.

Microcracking

Microcracking can be responsible for only the irreversible part of creep due to the fact that the cracking cannot be healed. Estimates are that microcracking is responsible for

10 to 20 percent of total creep deformation. The probable mechanism of creep is some combination of seepage, viscous flow and microcracking (39).

Factors That Influence Creep

Research has shown that there are many factors that influence the creep of concrete. Many of the factors that influence the creep of concrete are the same factors that influence the shrinkage of concrete. Factors known to influence creep are the compressive strength of the concrete, the age of the concrete when the sustained load is applied, the ambient humidity in which the concrete is stored, the aggregate used in the concrete, the temperature at which the concrete is stored, the type of cement used, the size of the specimen, and admixtures used in the concrete. The influence of these factors can be complex and some of the influences may simply be a manifestation of some other factor or factors. These factors will be discussed in more detail at this point.

Compressive strength

Neville (39) has shown that within a wide range creep is inversely proportional to the strength of concrete at the time of loading. More recently Ngab et al. (38) have found the same relationship to apply to high strength concrete. Timusk and Ghosh (46) conducted studies which indicate that creep does vary linearly with the stress-strength ratios providing that hydration does not occur after loading.

Age of the concrete at loading

The influence of the age of loading may be due to the increase in strength with age of the concrete. Neville (39) asserts that for ages greater than 28 days the influence of age at loading is negligible. The strength gain of concrete after 28 days is usually not large either. Parrott (43) has found that the creep of hardened cement paste decreases with age of loading even after hydration has ceased. Due to the restraining effect of aggregate on creep it does not necessarily follow that a small creep in the cement paste will be translated into a measurable creep in concrete. Timusk and Ghosh (46) found that the creep was influenced by the age of the concrete when loaded at less than 28 days due to hydration of the cement still taking place to a substantial degree.

Aggregate

The aggregate used in concrete has a substantial impact on the creep of concrete even though it undergoes very little, if any creep itself. The aggregate's influence is related to the restraint offered by the aggregate to the potential creep of the cement paste. This restraint is manifested in two ways--by the quantity of aggregate or the physical properties of the aggregates. Neville (39) asserts that increasing the cement content of concrete will, with a constant water content, cause an increase in creep due to a decrease in the amount of cement paste, but that an increase in the cement content could cause an increase in compressive strength and at least partially cancel out the increased creep. The physical property most important is the modulus of elasticity of the aggregate. The higher the modulus of elasticity the greater the restraint offered by the aggregates to the creep of the cement paste. The increase in creep that has been noted with more porous aggregates is probably due to the fact that more porous aggregates usually have lower moduli of elasticity. Tests of Hawaiian aggregates by Hamada (31) et al. have shown that concretes made with the more porous cinder aggregates show less creep than concretes made with the less porous basalt aggregates. In this study cylinders were loaded to 0.25 f'_c, 0.40 f'_c and 0.60 f'_c at a concrete age of 28 days. The specimens were moist cured for 7 days and then maintained in a controlled environment of 50 percent relative humidity at 73 degrees F. The cylinders were subjected to a uniaxial compression in accordance to ASTM C512 recommendations. The constant axial load was maintained by steel coil springs. The cinder aggregate concretes, however, showed greater overall strains due to creep, elastic and shrinkage (discussed previously) than the basaltic concretes did. Another facet of aggregates is in the effect gradation has on creep. Aggregate gradation was once thought to influence creep but now it is believed by Neville (39) that gradation is important in that it may influence the strength of the concrete. Li et al. (32) have found that for the same cement content and equal workability gap-graded concrete has less shrinkage and less creep than the corresponding continuously graded concrete. This study indicated higher compressive strengths and high moduli of elasticity for gap-graded than for continuously graded concrete.

Cement

The type of cement used influences creep only in that it influences the strength of the concrete. The fineness of cement influences creep but only because the strength at early concrete ages is influenced by the fineness of cement (1). Bennett and Loat (20) found that the creep of concrete was only slightly affected by the use of finer cement at the same water/cement ratio.

Ambient humidity

The relative humidity of the air in which concrete is stored is known to influence creep. Drying concrete creeps at a higher rate and has a greater ultimate creep than concrete that remains wet. This influence is large and can be 2 or 3 times greater for a concrete stored at a relative humidity of 50 percent than a concrete stored at 100 percent relative humidity. Myers and Slate (36) found the same result in experiments with unsealed specimens in a drying environment but found that in some cases creep was larger for specimens that were sealed. They attribute this apparently conflicting result to the fact that the sealed specimens do not necessarily have more water present that is in an uncombined form than for the unsealed specimens. A sealed specimen would not necessarily be the same as a specimen stored at 100 percent relative humidity because in the sealed specimen water that is combined cannot be replaced from the humidity of the surrounding air. Gamble and Parrott (29) found that the rate of development of drying creep was directly related to that of concurrent shrinkage. Gamble (28) has conducted tests with specimens stored at 70 degrees F and at a relative humidity of 50 percent and specimens stored outside subject to the extremes of temperature and humidity in Illinois. He found that the creep was not substantially different for the specimens stored outside but that the shrinkage was increased.

Temperature

This is the second environmental factor after ambient humidity that influences creep. Nasser and Neville (37) report that the rate of creep increases for increasing temperatures up to about 160 degrees F.

Size of specimen

Creep has been found to decrease with an increase in the size of a specimen (39). This may occur due to the fact that drying proceeds very slowly in the interior of larger concrete specimens and thus concrete on the surface is experiencing drying creep while the interior is experiencing creep at a different moisture condition than the surface.

Admixtures

Two admixtures are of particular interest, air entraining and superplasticizer admixtures. Ward (51) reports that air entrainment does not affect creep. Superplasticizers have thus produced conflicting results among researchers. Dhir and Yap (26), Brooks et al. (22) and Brooks et al. (23) report lower creep when the only change to the concrete is the addition of a superplasticizer. Brooks and Wainwright (21) and Neville and Brooks (41) report the same creep when a superplasticizer is used. Due to the many variables, such as the effect of changing mixes along with admixtures, and even to the different formulations of superplasticizers in use, such disparagent results are not unexpected.

Prediction of Creep and Shrinkage

Mathematical models for the prediction of creep and shrinkage appear to fall into two broad categories. The first is to define creep and shrinkage in terms of a number of multiplying factors where each of these factors represents the influence on creep and shrinkage of a specific intrinsic or environmental factor relative to some chosen reference state. These factors are generally considered to be independent of each other in these procedures. This type of procedure is represented by the three most commonly used mathematical methods for the prediction of shrinkage and creep in concrete: the ACI Committee 209 method (1), the CEB-FIP method (25), and the Bazant and Panula (BP Model) (12). The other approach has been to make use of rheological models. In some of these models, such as the Burgers model (33,39), an attempt is made to model some element of the time dependent behavior of specific components of the concrete. In other rheological models the elements of the model simply serve as a vehicle to predict the overall time-dependent behavior of the concrete without any attempt being made to relate them to any particular physical process (39).

The multiplying factors methods relates shrinkage and creep to variables that are more convenient to use in an engineering situation than a rheological model. These methods will be examined in more detail in the following portion of this chapter.

ACI Committee 209 Method

The ACI Committee 209 method (1) is in the form of equations with modifiers for conditions different from standard. The equations are for creep and for normal weight, and light weight concrete.

Creep. The creep coefficient C_t is defined by the following equation:

$$C_t = C_u K_t K_a K_h K_{th} K_s K_f K_e$$

The coefficients for this equation are defined below.

Ultimate creep coefficient, C_u, is the ratio of the ultimate creep strain to the initial strain. The purpose is to relate the creep strain at a specified time to the ultimate creep strain by the ultimate creep coefficient, C_u. The value of C_u can vary widely. In the ACI Committee 209 review, C_u was found to be in the range 1.30 to 4.15, with an average value of 2.35. This average value should be assumed only in the absence of more exact data for the concrete to be used.

Time under load coefficient, K_t

$$K_t = \frac{t^{0.6}}{10 + t^{0.6}}$$

where t = time in days after application of load (K_t = 0.44, 0.60, 0.69, 0.78, and 0.90 for t = 1 month, 3 months, 6 months, 1 year, and 5 years, respectively).

Age when loaded coefficient, K_a

$K_a = 1.25 t_i^{-0.118}$ · for moist-cured concrete

or

$K_a = 1.13 t_i^{-0.095}$ for steam-cured concrete where t_i = age of concrete in days when load is first applied (K_a = 1.00, 0.95, 0.83, and 0.74 for moist-cured concrete loaded at 7, 10, 30, and 90 days, respectively; K_a = 1.00, 0.90, 0.82, and 0.74 for steam-cured concrete loaded at 1 to 3, 10, 30, and 90 days, respectively).

Relative humidity coefficient, K_h

$K_h = 1.27 - 0.0067H$ for $H > 40\%$

where H = relative humidity in percent

(K_h = 1.00, 0.87, 0.73, and 0.60, for $\leq$ 40, 60, 80, and 100% relative humidity).

For $H < 40\%$, values higher than 1.0 shall be used for K_h.

Minimum thickness of member coefficient, K_{th}

K_{th} = 1.00 for 6 in or less, and 0.82 for 12 in (1 in = 25.4 mm)

For thickness of members other than 6 in or volume-- surface ratio other than 1.5 in details can be found in Reference 1.

Slump of concrete coefficient, K_s

$K_s = 0.82 + 0.0675\ s$

where s is the observed slump in inches.

Fines coefficient, K_f

$K_f = 0.88 + 0.0024f$ for $f \leq 50\%$

f = ratio in percent by weight of fine aggregate to total aggregate

Air content coefficient, K_e

$K_e = 0.46 + 0.09a$

where a is the air content in percent.

The cement content need not be taken into account for concrete with cement contents between 470 and 750 lb/yd^3 ($1\ \text{lb/yd}^3 = 0.593\ \text{kg/m}^3$).

Shrinkage. The shrinkage strain at any time t is given by the equation:

$$\varepsilon_{sh} = \varepsilon_{shu} S_t S_h S_{th} S_s S_f S_e S_c$$

where the coefficients are given below.

Ultimate shrinkage strain, ε_{shu}, which represents the expected shrinkage strain when the concrete has ceased all shrinkage. The value of ε_{shu} can vary widely. In the ACI Committee 209 review, ε_{shu} was found to be in the range 0.000415 to 0.00107, with mean values of 0.00080 for moist-cured concrete or 0.00073 for steam-cured concrete. These average values should be assumed only in the absence of more exact data for the concrete to be used.

Time of shrinkage coefficient, S_t,

At any time after age 7 days, for moist-cured concrete,

$$S_t = \frac{t}{35 + t}$$

where t = time in days from age 7 days

(S_t = 0.46, 0.72, 0.84, 0.91, and 0.98 for t = 1 month, 3 months, 6 months, 1 year, and 5 years, respectively).

Relative humidity coefficient, S_h

$S_h = 1.4 - 0.01H$ for $40 < H < 80\%$

or

$S_h = 3.0 - 0.03H$ for $80 < H < 100\%$

where H = relative humidity in percent

(S_h = 1.00, 0.80, 0.60, 0 for ≦ 40, = 60, = 80, and = 100% relative humidity).

Minimum thickness of member coefficient, S_{th}

S_{th} = 1.00 for 6 in or less and 0.84 for 9 in (1 in = 25.4 mm)

See Reference 1 for other sizes.

Slump of concrete coefficient, S_s

$S_s = 0.89 + 0.41\ s$

where s is the slump of concrete in inches.

Fines coefficient, S_f

S_f = 0.86 for 40%, 1.00 for 50%, and 1.04 for 70% fines by weight.

Air content coefficient, S_e

S_e = 0.98 for 4%, 1.00 for 6%, and 1.03 for 10% air

Cement content factor, S_c

S_c = 0.87 for 376 lb/yd^3, 0.95 for 564 lb/yd^3, 1.00 for 705 lb/yd^3, and 1.09 for 940 lb/yd^3 (1 lb/yd^3 = 0.593 kg/m^3).

More details and application of this model can be found in Reference 1.

C.E.B.-F.I.P. Model

The procedure recommended by the C.E.B. (25) for the prediction of creep is based on linear creep theory which leads to the conclusion that, for a given concrete under constant environmental conditions and subjected to a constant sustained load, the creep strain is linearly related to the instantaneous elastic strain. This assumption is generally true for the range of stress levels encountered in most structures at working conditions.

Creep. The following expression is given for creep strain ε_{cr}:

$$\varepsilon_{cr} = \frac{f_c}{E_{b28}} \phi_t$$

where: f_c = constant sustained concrete stress

E_{b28} = secant modulus of elasticity of the concrete at 28 days.

ϕ_t = creep coefficient.

The tangent modulus of elasticity for normal aggregate concretes at any time may be calculated as follows:

$$E_c(t) = 66{,}000\sqrt{f_c'(t)}$$

with: $E_c(t)$ and $f_c'(t)$ both in N/cm^2

$$\text{or } E_c(t) = 79{,}400\sqrt{f_c'(t)}$$

with: $E(t)$ and $f(t)$ both in Psi.

where: $f_c'(t)$ is the cylinder compressive strength at time t. The ratio of the compressive strength at an age of t days to the 28 day compressive strength, as recommended by the C.E.B. for normal aggregate concrete, is presented graphically in Reference 25.

The secant modulus of elasticity is taken as being 90 percent of the tangent modulus of elasticity.

The creep coefficient reflects the quality of the concrete as well as the expected environmental conditions. It is given as the product of five partial coefficients:

$$\phi_t = K_c K_d K_b K_e K_t$$

Each of these coefficients is presented in the form of a graph, which can be found in Reference 25.

K_c: K_c depends on the relative humidity and is thus a reflection of the environment.

K_d: K_d reflects the degree of hardening of the concrete at first loading. The graphs given in Reference 25 can be used to determine K as a function of age of loading, if the concrete hardens at an average concrete temperature of 20 degrees C and also if sufficient protection against excessive loss of moisture has been provided. If hardening takes place at a temperature other than 20 degrees C, the corresponding degree of hardening may be used instead of age by the graphs in Reference 25 to determine K. The degree of hardening is calculated from:

$$D = \Sigma\Delta_t(T + 10)$$

where: D = degree of hardening

Δ_t = number of days during which hardening has taken place at T°C.

K_b: K_b expresses the dependence of creep on the composition of the concrete mix and is given as a function of the water/cement ratio as well as the cement content.

K_e: The dependence of creep on member size is given by this factor. It is expressed as a function of the theoretical thickness, where the theoretical thickness, given by twice the ratio of the cross-sectional area to the perimeter exposed to the atmosphere, in turn describes the member size and shape.

K_t: K_t describes the time-dependence of creep. The effect of member size on the rate of creep is reflected here by presenting K as a function of the theoretical thickness as well as time.

Shrinkage. As for the case of creep, the shrinkage of concrete at constant environmental conditions is expressed by the C.E.B. as the product of five multiplying factors, thus:

$$\varepsilon_{sh} = \varepsilon_c K_b K_e K_p K_t$$

K_b and K_t are taken to be the same as for creep. It is, however, important to realize that in the evaluation of K for shrinkage, the time is considered to be the elapsed time since the end of curing.

ε_c: This is the basic shrinkage strain and is given as a function of relative humidity.

K_e: This factor reflects the dependence of shrinkage on the theoretical thickness of the member.

K_p: K_p takes account of the restraining effect of the longitudinal steel reinforcement on the shrinkage strain. It is given by the following expression:

$$K_p = \frac{100}{100 + np}$$

where: $n = 20$ with regard to creep and

p = percentage longitudinal steel

These expressions apply to creep and shrinkage where the temperature and relative humidity remain constant.

Bazant-Panula Model (BP Model)

The BP model has been extensively developed by Bazant (9,10,11,12), Bazant and Panula (17), Bazant and Kim (14,15), Bazant and Najjar (16), Bazant and Asghari (13), Bazant and Wu (19), Bazant, Tsubak and Celep (18), and Madsen and Bazant (35). The BP model is a development of the double power law (12) for creep. The BP model, as simplified by Bazant and Panula (17), will be presented. The formulas that this model is based on are as follows:

Shrinkage strain:

$$\varepsilon_{sh} = \varepsilon_{sh\infty} K_h S \left(\frac{\hat{t}}{\tau_{sh}}\right), \quad \hat{t} = t - t_0$$

Strain due to unit stress:

$$\varepsilon = J(t, t') = \underbrace{\frac{1}{E_0}}_{\substack{\text{initial}\\\text{strain}}} + \underbrace{A(t')F(\bar{t})}_{\text{basic creep}} + \underbrace{k_h' P(\Delta_d) B(t') f\left(\frac{\hat{t}}{\tau_{sh}}\right)}_{\text{drying creep}}$$

where

$J(t,t')$ = creep (compliance) function

= strain at time t (in days) caused by a unit uniaxial stress sustained since time t' (in days)

$\bar{t} = t - t'$ = stress duration

$\Delta_d = t' - t_0$

= time delay from the start of drying to the application of the load at time t'

$\hat{t}$ = drying duration

k_h and k_h' = functions of ambient humidity

τ_{sh} = shrinkage-square halftime which is proportional to the size square. τ_{sh} involves the dependence on size and on disffusivity as a function of age.

Note that S, A, F, P, B, and f are functions of the indicated variables.

Creep. The prediction of creep is divided into two terms, basic creep and drying creep. The compliance function for elastic, basic creep and drying creep is:

$$J(t,t') = \frac{1}{E_0} + C_0(t,t') + C_d(t,t',t_0)$$

Basic Creep--the double power law is used to define the basic creep $C_o(t,t')$ as a function of t and t':

$$C_0(t,t') = \frac{\psi_1}{E_0}(t'^{-m} + \alpha)(t - t')^n$$

where:

$$\psi_1 = 0.3 + 15(f'_c)^{-1.2}$$

$$m = 0.28 + 1/(f'_c)^2$$

$$n = 0.115 + 0.0002(f'_c)^3$$

$$\alpha = 0.05$$

where f'_c is in ksi.

The above is applicable to normal weight and normal strength concretes for load durations $t - t' \geq 1$ day.

Drying Creep--The function for drying creep term $C_d(t,t',t_0)$ is:

$$C_d(t,t',t_0) = \frac{\bar{\psi}d}{E_0} K_h' t'^{-m/2} S_d(t,t')$$

$$K_h' = 1 - h^{1.5}$$

$$\bar{\psi}d = \left(1 + \frac{t' - t_0}{10\tau_{sh}}\right)^{-\frac{1}{2}} \psi_d (10^6 \varepsilon_{sh\infty})$$

$$S_d(t,t') = \left(1 + \frac{3\tau_{sh}}{t - t'}\right)^{-0.35}$$

h = relative humidity of the environment

Shrinkage. The terms in the formula for shrinkage strain are defined as follows:

$$S\left(\frac{\hat{t}}{\tau_{sh}}\right) = \sqrt{\frac{\hat{t}}{\tau_{sh} + \hat{t}}}$$

$k_h = 1 = h^3$ for $h \leqq 0.98$;

$k_h = -0.2$ for $h = 1.00$

S is a function of $\hat{t}$ and τ_{sh}.

For the effects of size, diffusivity and age, the earlier BP formulas are simplified as:

$$\tau_{sh} = \frac{(k_s D)^2}{C_1(t_0)} \; ; \; D = 2\,\frac{v}{s} \; ; \; C_1(t_0) = 2.4 + \frac{120}{\sqrt{t_0}}$$

in which

t = time (in days), representing the age of concrete

t_0 = age when drying begins

$\hat{t}$ = duration of drying

ε_{sh} = shrinkage strain

$\varepsilon_{sh\infty}$ = ultimate shrinkage

h = relative humidity of the environment

τ_{sh} = shrinkage square halftime

$C_1(t_0)$ = coefficient proportional to drying diffusivity at age t_0

D = effective cross section thickness in mm

v/s = volume-to-surface ratio in mm

k_s = shape factor which equals 1.0 for a slab, 1.15 for a cylinder, 1.25 for a square prism, 1.30 for a sphere, 1.55 for a cube.

A full description of all the preceding terms and the model can be found in Reference 17. The BP model has one feature not found in the other models discussed. It is the provision to improve the prediction if a shorttime value is known. The method for doing this is described in Reference 56.

Rheological Models

Burgers Model

The Burgers model (1,48) is a rheological model in which the elements of the model represent physical portions of the creep phenomenon. The Burgers model is made up of two springs and two dashpots as shown in Figure 2.3.a. The spring E_1 represents the elastic response, the dashpot η_3 the permanent creep, and the spring E_2 and dashpot η_2 in parallel represent the recoverable creep. The Burgers model cannot be used to predict shrinkage.

When a sustained load is applied to the Burgers model, an instantaneous strain (ε_0) takes place at spring E_1 followed by a time dependent strain (ε_d) of the parallel spring E_2 and the dashpot η_2. Together with these two strains is a third strain (ε_c) that results from the dashpot η_3 and is linear dependent with time. The time dependence of these strains are shown in Figure 2.3.b. On removal of the sustained load an instantaneous recovery, strain (ε_0), takes place followed by a time dependent recovery, strain (ε_d), that exponentially approaches a straight line. The part of the strain (ε_c) attributed to the dashpot η_3 is not recovered and considered a permanent set. A typical strain versus time curve for the Burgers model is shown in Figure 2.3.c.

The Burgers model has the following equations for strain:

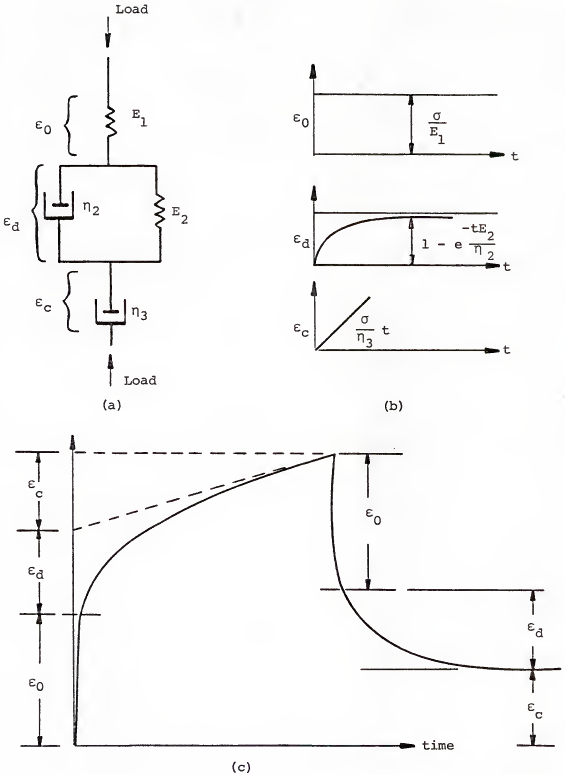


Figure 2.3. Burgers Model (a) Model Components, (b) Component Strain Time Relationships, (c) Combined Strain Time Curve.

$$\varepsilon_0 = \frac{\sigma}{E_1}$$

$$\varepsilon_d = \frac{\sigma}{E_2}\left(1-e^{-t/\tau}\right) \quad \text{where:} \quad \frac{1}{\tau} = \frac{E_2}{\eta_2}$$

$$\varepsilon_c = \frac{\sigma}{\eta_3}\, t$$

where: t = the time since loading

σ = applied sustained stress

E_1, E_2, E_3 = the modulus of elasticity of the springs

η_1, η_2, η_3 = viscosity of the dashpots

Indirect Tensile Creep

Constant stress indirect tensile tests produce a theoretical stress distribution as shown in Figure 2.4. Neville (39) reports that there has not been enough tests in tension for any systematic quantitative information to be derived. None of testing reported by Neville (39) used the indirect tensile creep method used herein.

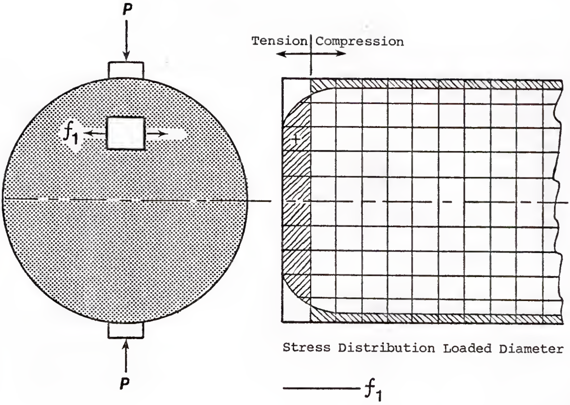


Figure 2.4. Split Cylinder Test for Tensile Strength with Stress Distribution Across the End of the Cylinder.

CHAPTER III
MATERIALS, SPECIMENS AND INSTRUMENTATION

Introduction

To provide data concerning the influence of different coarse aggregates on the creep behavior of concrete a series of concrete mixtures using different coarse aggregates were prepared. The specimens were instrumented such that compressive creep and indirect tensile creep could be measured. Companion shrinkage studies were carried out for each coarse aggregate.

Materials

Cement

The cement used was a Type II Portland Cement (Broco brand). Details of the analysis of the cement performed by the Florida Department of Transportation are shown below with ASTM C150 specifications shown in parenthesis after the analytical data:

Blaine air permeability test: 3950 cm^2/g (2800 cm^2/g min.)
Normal consistency: 25.0%
Soundness (autoclave expansion): 0.060% (Max. 0.80%)
Time of setting (Gilmore): Initial 3:05 hours (Min. 1 hr.)
Final 5:10 hours (Max. 10 hrs.)
Compressive strength (using 2 inch cube specimens)
3 days: 2358 psi (Min. 1000 psi)
7 days: 3526 psi (Min. 1800 psi)

Chemical analysis:

Loss on ignition, max. %	2.6	(Max. 3.0)
Insoluable residue, max. %	0.37	(Max. 0.75)
Sulphur trioxide, %	2.7	(Max. 3.0)
Magnesium oxide, max. %	0.52	(Max. 5.0)
Tricalcium aluminate, max. %	7.6	(Max. 8.0)
Silicon dioxide, min. %	23.6	(Min. 21.0)
Aluminum oxide, max. %	4.9	(Max. 6.0)
Ferric oxide, max. %	4.3	(Max. 6.0)
Tricalcium silicate, max. %	39.2	---

Aggregates

Four coarse aggregates were used. Three of the coarse aggregates (Miami Oolite, Brooksville and Ft. Thompson) are commonly used Florida aggregates and one (Calera) was from near Birmingham, Alabama. The three Florida aggregates were a porous limestone, whereas the Calera aggregate was a relatively nonporous limestone. Details of the aggregates are shown in Table 3.1. Gradation charts for the Miami Oolite, Brooksville, Ft. Thompson and Calera are shown in Figures 3.1, 3.2, 3.3 and 3.4, respectively.

The fine aggregate used for all mixtures was a sand from Goldhead, Florida. The gradation for this sand is shown in Figure 3.5. Details of the fine aggregate are shown below:

Specific gravity:	2.38
Absorption:	0.34%
Fineness modulus:	2.16

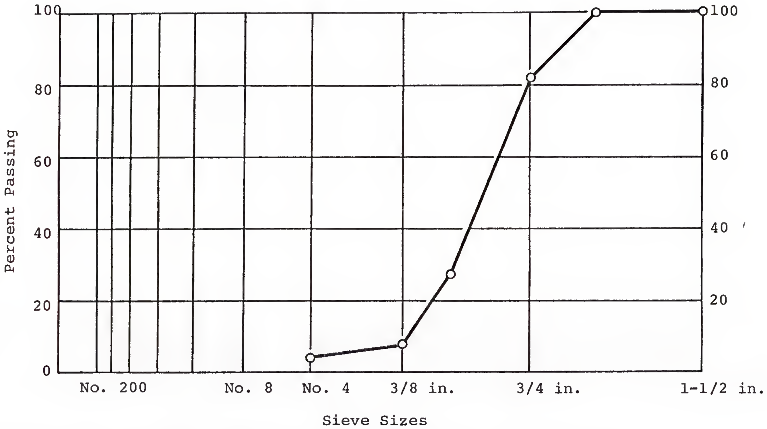


Figure 3.1. Gradation Chart for the Miami Oolite Coarse Aggregate.

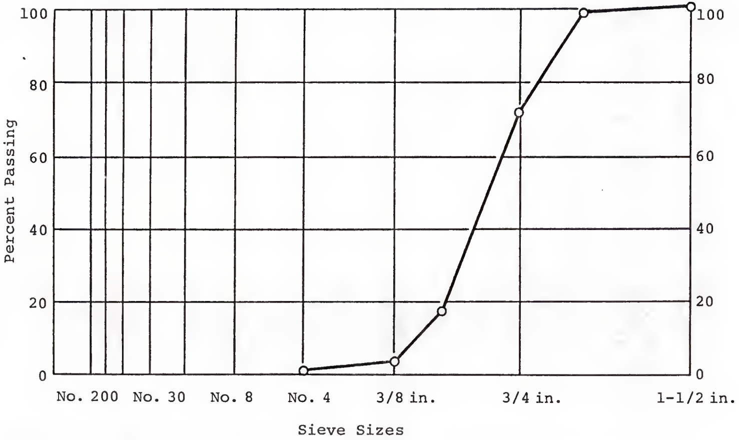


Figure 3.2. Gradation Chart for the Brooksville Coarse Aggregates.

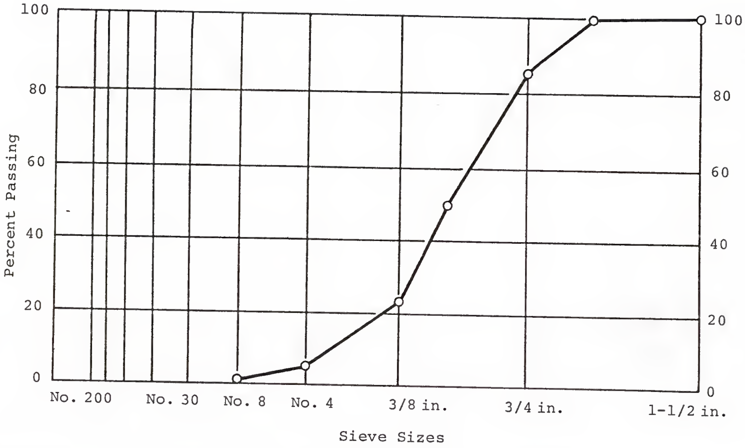


Figure 3.3. Gradation Chart fot the Ft. Thompson Aggregate.

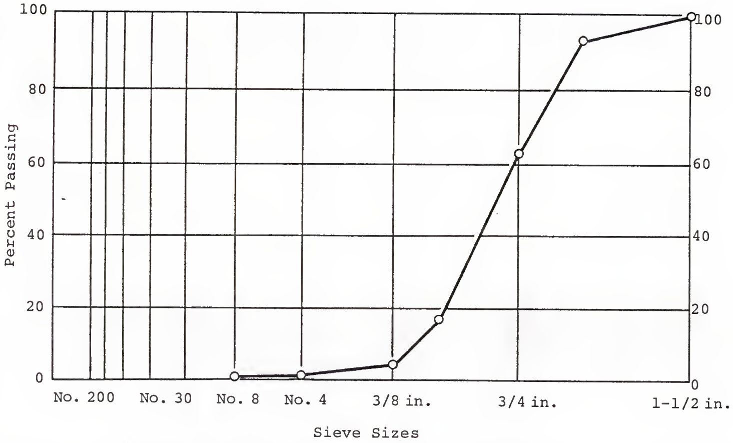


Figure 3.4. Gradation Chart for the Calera Coarse Aggregate.

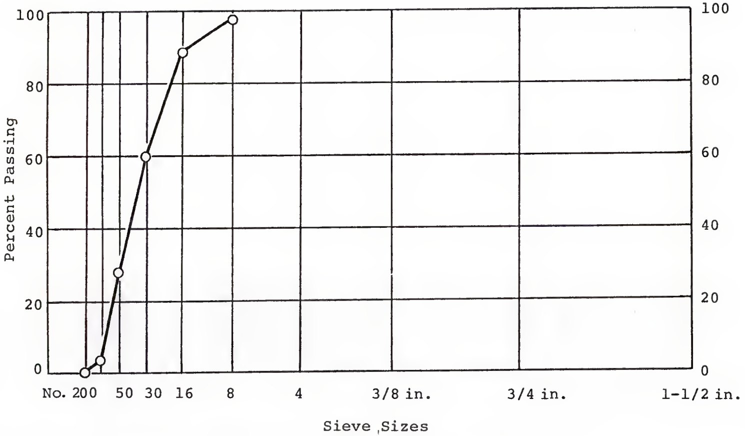


Figure 3.5. Gradation Chart for Goldhead Sand Fine Aggregate.

Table 3.1. Physical Properties of Coarse Aggregates.

Property	Coarse Aggregate Miami Oolite	 Brooksville	 Ft. Thompson	 Calera
Specific Gravity (SSD)	2.42	2.52	2.45	2.78
Absorption (%)	4.70	3.26	3.99	0.30
Unit Weight (lbs/ft^3)	78.4	89.2	85.8	98.8
Los Angeles Abrasion Ratio (%)	33	33	30	18

Admixtures

The only admixture used was Darex brand of air entraining agent. This admixture was used in all the concrete mixtures.

Manufacture of Specimens

Four batches of concrete were prepared using each of the four aggregates. The concrete was designed to meet the following specifications:

Water/cement ratio: 0.41

Slump: 0 to 4 inches

Cement Content: 7 bags per cu. yd.

Minimum 28 day f'c: 3600 psi

Maximum 28 day f'c: 6000 psi

The concrete mixtures had the properties shown in Table 3.2. The mixtures were prepared so that the absolute volume of each ingredient remained the same for each

Table 3.2. Concrete Properties.

	Coarse Aggregate			
Property	Miami Oolite	Brooksville	Ft. Thompson	Calera
Slump (inches)	2.75	4.0	2.0	1.0
Actual Air Content (%)	4.0	4.0	4.0	4.0
Calculated Air Content (%)	3.78	3.37	4.95	1.98
Calculated Unit Wt.	138.5	140.1	138.8	144.4
Actual Unit Wt.	138.6	140.8	137.2	147.6
Components lbs/yd^3				
Water	269.8	269.8	269.8	269.8
Cement	658.0	658.0	658.0	658.0
Coarse Agg. (Dry)	1510.1	1572.5	1528.0	1734.7
Fine Agg. (Dry)	1231.4	1231.4	1231.4	1231.4
Water/Cement Ratio	0.41	0.41	0.41	0.41
Air Entraining Agent (Darex) ml	80	80	80	80

concrete batch. The mixtures were all prepared in a rotary type mixer with a capacity of 25 cubic feet. The concrete batches were all prepared in 25 cubic foot batches. The aggregates and cement were added and mixed for 3 minutes with approximately 80% of the mixing water. After 3 minutes the mixer was shut off and the concrete was allowed to rest for 2 minutes. After 2 minutes the mixing was continued and the remaining water together with the air entraining agent was added to the mixture. After mixing, a standard slump cone test (4), entrained air (5) and unit weight (3) tests were performed. The concrete mixture was then placed in 6" x 12" carbboard cylinder molds. The molds were placed on a vibrating table and vibrated while the cylinders were being filled.

Three pairs of gage seats were cast into the cylinders as needed for shrinkage and compressive creep tests. The pairs of gage seats were maintained at the desired spacing of 8 inches by using an aluminum spacer bar that held each pair of gage seats in the proper position on the outside of the cylinder molds. Maintaining a precise distance between pairs gage points was necessary because the Whittemore gage had a total travel of only 0.2 inches. The gage points were screwed into the gage seats at a concrete age of three days for the Miami Oolite concrete and at seven days for the other concrete mixtures. This change occurred because the gage seats in the other concretes loosened when an attempt was made to screw the gage points in at three days. Gage seats that were loosened, were removed, coated with epoxy and placed back into the concrete cylinder.

Test Apparatus and Instrumentation

Compressive Creep

Test apparatus

Load frames designed to maintain a constant level of stress on the specimens were constructed as shown in Figure 3.6. Each frame consisted of two heavy steel end plates (24" x 24" x 2") connected by three steel rods. Four cylinder specimens were placed end-to-end between the heavy end plates.

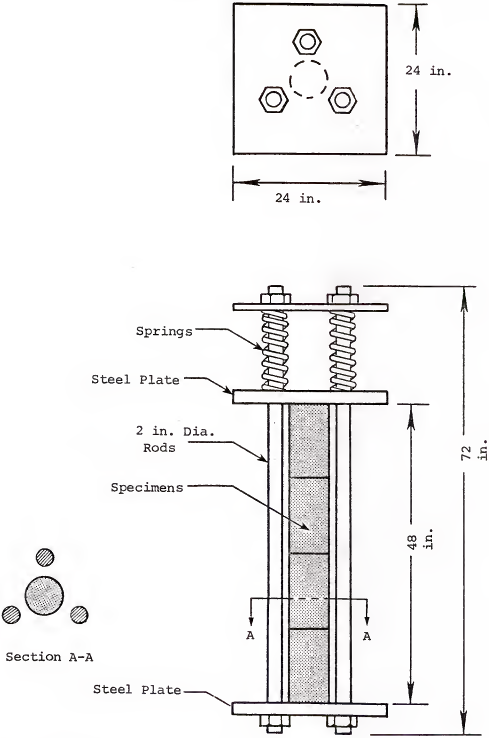


Figure 3.6. Test Frame for Creep Test.

The load was applied by nuts at the end of the steel rods. The springs between the nuts and rods helped to maintain a constant load (the spring constant was 9000 lbs/in) while the specimens deformed.

The four cylinder specimens were cemented together into a single unit by the use of a concrete epoxy (Sikadur 31 Hi-Mod Gel). The cylinders were placed in an alignment frame one day before the compressive creep test was begun and the cylinders were than cemented together end to end. The two alignment frames used consisted of a 6 inch channel welded in a vertical position to a steel base plate (Figure 3.7). The alignment frame was placed inside the compressive creep testing frame with the channel section vertical. The cylinders were then placed in the frames with the epoxy applied to the ends of the cylinders that would abut each other and the bottom of the bottom cylinder where it made contact with the bottom plate of the creep frame; the top of the top cylinder was not epoxied. The bottom was epoxied to assure a vertical alignment of the cylinders. If the cylinder surfaces were not sufficiently smooth to assure an even contact, they were ground smooth before epoxing. Once the cylinders were in the frame a 2 x 2 inch angle secured the cylinders firmly to the channel. The top plates and springs were lowered onto the cylinders and kept there until the epoxy set. The ends of the cylinders that were to bear against the heavy end plates

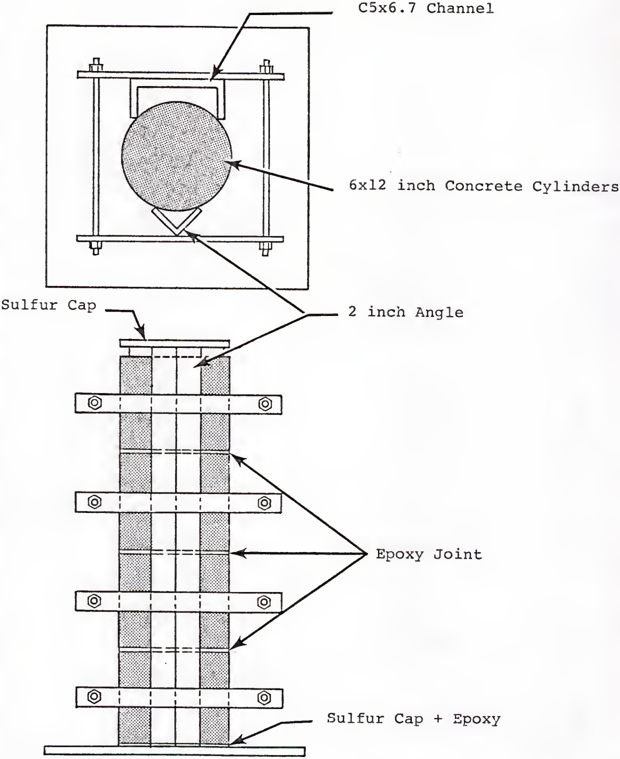


Figure 3.7. Alignment Frame

were either capped with sulfur or epoxied for a smooth bearing surface.

The load was applied by placing the frame in a Rhiele 400 kip testing machine and loading the upper plate that was bearing against the springs until the desired load was obtained and the nuts were tightened to maintain the load. The load was applied so as to keep the two upper plates of the frame parallel and thus apply an even load, and the deformations were monitored by dial gage.

Instrumentation for Compressive Creep

The test specimens that were placed in the frames had three pairs of gage points seats spaced at 8 inches cast into each cylinder. A dial extensometer (Whittemore gage) with a 0.0001 inch precision was used to monitor the deformations over the 8 inch gage length.

Indirect Tensile Creep

Test apparatus

Indirect tensile creep measurements were performed by loading the cylindrical concrete specimens on their side as was done in the indirect splitting tensile test (7). A constant load of either 25 percent or 50 percent of f_t was maintained on the cylinders. The testing was performed by stacking the cylinders on their sides four high and applying a load to the top cylinder (Figure 3.8). One-eighth inch wood strips were placed between the cylinders and the

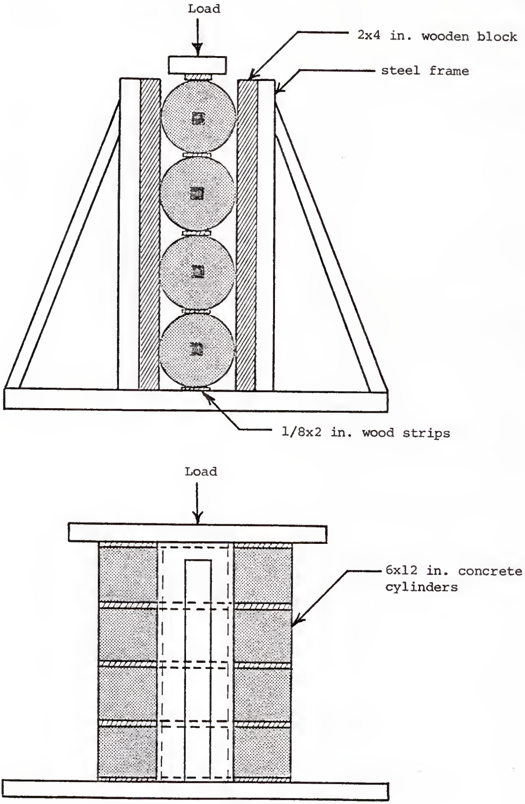


Figure 3.8. Indirect Tensile Creep Lateral Support Frame.

loading plates. A steel frame was used to hold the cylinders vertically and keep them from rolling when the load was applied. The load was applied by either of two methods--placing the entire setup (cylinders and support frame) in a mechanical Rheile testing machine or by using the test frames from the compressive creep test. When the test frames from the compressive creep test were used, only two of the three springs were used, a swivel head was placed between the loading plate and the bar that bore against the cylinders and load was monitored by a load cell as shown in Figure 3.9. The load was applied by use of a hand operated hydraulic jack and the nuts tightened when the desired load was obtained.

Instrumentation for Tensile Creep

Strain measurements were recorded using Micro-Measurement's EA-06-20CBW-120 one-half inch long strain gages. The gages were attached by epoxy along the horizontal axis at the center of the ends of each cylinder. The gages were mounted perpendicular to the applied load. Strain readings were taken with a switch-and-balance unit using the one-quarter bridge circuit of a Vishay brand portable strain indicator unit.

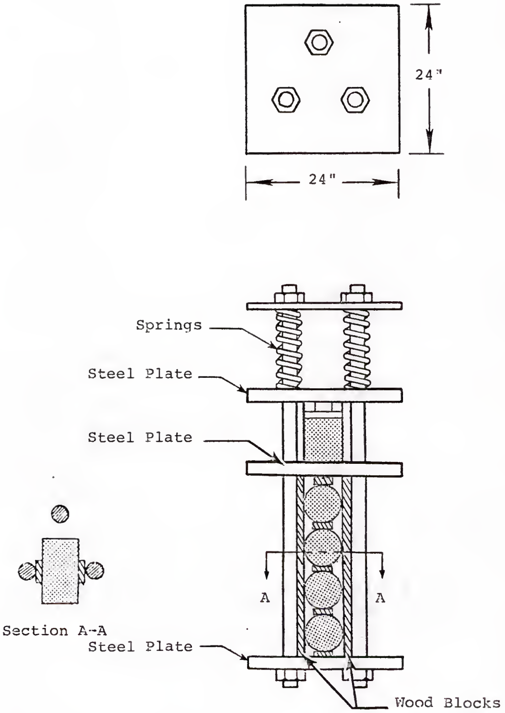


Figure 3.9. Creep Frame Setup for Indirect Tensile Creep Test

CHAPTER IV
TEST PROCEDURES

Introduction

The testing of the specimens was divided into two parts: compressive creep testing and the indirect tensile creep testing. The compressive and tensile creep testing consisted of placing test cylinders in testing frames and loading the cylinders to a constant stress and monitoring the deformations. Because the compressive creep deformation included shrinkage, additional cylinders not subjected to a load were monitored for shrinkage. The testing period was for a minimum of one year of age of the concrete for the compressive creep test and 48 hours for the tensile creep test.

Compressive Creep

In addition to the type of aggregate used, principal variables in this study were the magnitude of the constant stress applied to the specimens, the environment in which the specimens are stored, and the age at which the load is first applied to the specimens. The various combinations of variables for the compressive creep tests are shown in Table 4.1. The stress levels represent fractions of concrete strength applied as sustained loads. The moist environment conforms to ASTM requirements (8) of greater than 98%

Table 4.1. Combinations of Variables for Compressive Creep Tests.

Mix	Moist Environment			Air Conditioned Environment		
	Stress Level[1]			Stress Level[1]		
	0.25 F'_c	0.50 f'_c	0.5 f'_c	0.25 f'_c	0.50 f'_c	Other
Miami Oolite		28 da.[2]		28 da.	3 da. 28 da. 90 da.	
Ft. Thompson Aggregate		28 da.		28 da.	7 da. 28 da.	28 da.[3]
Brooksville Aggregate	28 da.	28 da.	28 da.	28 da.	7 da. 28 da. 90 da.	28 da.[4]
Calera Aggregate		28 da.		28 da.	7 da. 28 da.	28 da.[5]

[1] f'_c = Compressive Strength of Concrete

[2] Times refer to age at first loading

[3] 0.40 f'_c

[4] 0.50 f'_c

[5] 0.40 f'_c
0.33 f'_c
0.25 f'_c

relative humidity and 73 ± 3 F temperature. The environment was an air conditioned room maintained at 70 ± 3 F temperature with a 60% ± 15% relative humidity. Small variabilities in the environments were handled by storing the specimens for all mixtures side-by-side, such that they all experienced the same variations. Each of the 24 cases shown in Table 4.1 represents one testing frame, containing four replicate specimens for that case.

After casting the cylinder specimens were covered with plastic and left for approximately 24 hours before the cardboard cylinder molds were stripped off and the cylinders placed in either the moist room or the air-conditioned environment where they would remain throughout the testing period. No other curing was done for the specimens stored in the air conditioned room.

As soon as the cylindrical specimens were sufficiently strong (3 days for the mixture made with Miami Oolite and 7 days for the others) the stainless steel gage seats were installed by screwing them into the brass inserts that were cast into the cylinders. Monitoring of shrinkage deformations was begun as soon as the seats were installed.

Immediately prior to installing specimens in the creep frames, three to four cylinder specimens were tested in direct compression (2) to determine the compressive strength. Two of these cylinders were also tested in accordance with

ASTM standards (6) to obtain a stress-strain relation. This was repeated at the conclusion of the one year test period. These tests were performed for each of the four mixes and for specimens stored in each of the two environments, at 3 or 7 days, 14 days, 28 days, 60 days, 90 days, 180 days, 270 days, and 365 days.

Creep and shrinkage deformations were monitored until the concrete mixture was at least one year old regardless of the age of the mix when the constant stress was applied. During the first few hours the deformations were recorded hourly and thereafter for about 2 weeks they were recorded daily. When subsequent deformations showed little change, the deformations were recorded three times weekly, then twice weekly, then once weekly and finally once every two weeks. A change in subsequent deformations of 0.0004 inches was considered a significant change. At the end of the one year period selected frames had the load removed and the creep and elastic strain recovery was monitored.

Indirect Tensile Creep

Indirect tensile creep tests were conducted using different levels of constant stress applied for at least 48 hours. These tests were performed at various ages for each mix and at two stress levels. The cases are shown in Table 4.2. Four replicates were used and only specimens stored in the air conditioned environment were tested. In order to

Table 4.2. Combinations of Variables for Indirect Tensile Creep Study.

Mix	Stress Level[1]	
	$0.25\ f_t$	$0.50\ f_t$
Miami Oolite	28 day	3 day 28 day
Ft. Thompson Aggregage	28 day	7 day 28 day
Brooksville Aggregate	28 day	7 day 28 day 90 day
Control Aggregate	28 day	7 day 28 day

---

[1] f_t = Tensile strength of concrete.

choose the levels of loading for the indirect tensile creep tests, indirect tensile strength tests (7) were performed for each mix, for the same ages as the tensile creep tests. Four replicates were tested.

CHAPTER V
ANALYSIS OF DATA

Limitations of the Scope of Study

Several limitations were placed on this study that tend to limit the scope of the analysis. The first limitation is that only one batch of concrete was made with each aggregate. If other factors, such as the temperature of batching (this could be a seasonal effect), and the training of the batching crew, can be assumed equal then the comparisons that follow are valid without reservation. Another limitation is that the load from frame to frame may vary, which would also affect the measured strains. These limitations are not considered to be large and the following comparisons are considered valid.

Strength of Concrete

Compressive Strength

Compressive strengths (f'_c) were measured for the concrete specimens during the one year test period. Tables of all compressive strength data can be found in Appendix D. The plot of the compressive strength (psi) versus the age of the concrete (days) is shown in Figure 5.1.(a). for specimens stored in the air conditioned environment. The concretes made with Miami Oolite and Brooksville aggregates follow a typical strength gain with time, where there is a steep initial gain during the first 60 days or so and then a flattening of the curve with increasing age. The concrete made with the Ft. Thompson aggregate shows a similar relationship but the increase is only about half that of the Miami Oolite or Brooksville concretes. The concrete made with the Calera aggregate shows only a very slight strength gain during the first 60 days and actually shows a decrease in strength after this time. The specimens of Calera concrete that were tested at 365 days were carefully examined after failure and it was noted that the bond of the cement paste to the coarse aggregate particles had failed. This bond failure had not been noted in previous compressive tests with the Calera concrete.

Figure 5.1.(b). shows the compressive strength (psi) versus the age of the concrete (days) for the concrete stored in the moist room. All the concrete mixtures showed similar properties when stored in the moist room. The Calera concrete in particular did not show the decrease in strength after 60 days that was shown in the tests with samples that had been stored in the air conditioned room. The Calera concrete showed a very strong increase in strength with time. Inspection of the failed compressive specimens did not show the bond separation that characterized the specimens from the air conditioned storage. A hypothesis for this difference between the moist room stored and the air conditioned

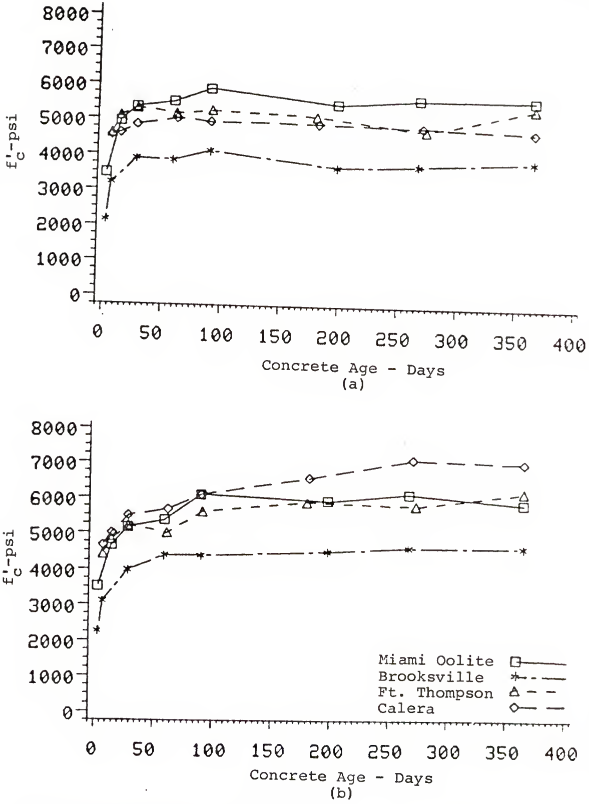


Figure 5.1. Influence of Aggregate on the Compressive Strength (f'_c) of Concrete Stored in the (a) Air Conditioned Room and (b) the Moist Room.

stored specimens is that due to the relatively low water cement ratio the specimens stored in the air conditioned room had ceased hydration of the cement relatively quickly due to the evaporation of available water and due to this fact less cement gel was created to fill the voids and increased shrinkage strains result. This situation does not exist in the moist room where the relative humidity is kept at 100%. This increased shrinkage strain is more severe on the bond with the Calera aggregate due to its hard nonporous nature. The Florida aggregates, being very porous, offer a better bond plus the possibility that water in these voids could be available for hydration.

Five equations were fitted to the compressive strength data and details are provided in Table D.6 in Appendix D. The Gompertz equation ($f'_c = Ae^{B/t}$), where A and B were found by linear regression and t is the age of concrete, generally had the best fit and plots of the equation with the experimental compressive strengths is provided in Figures D.1 through D.8 in Appendix D.

Table 5.1 shows the relationship between the consistency of the concrete mixtures, expressed in inches of slump, and the mean 28 day tensile and compressive strengths. The tensile strength for air conditioned room specimens and the compressive strengths for specimens stored in the moist room varied directly with the slump. The compressive strength for the specimens stored in the air conditioned room did not

Table 5.1. Consistency of Concrete and 28 Day Strengths.

Aggregate	Slump (inches)	f'_c (psi)		f_t (psi)
		Air Conditioned	Moist	Air Conditioned
Calera	1.00	4832	5487	497
Ft. Thompson	2.00	5292	5203	389
Miami Oolite	2.75	5340	5152	383
Brooksville	4.00	3880	3966	260

exhibit any cause effect relationship with the consistency of the concrete. The most likely cause of this decrease of strength with increased consistency is segregation of the aggregate. The concrete cylinders were consolidated by external vibration while the cylinder molds were being filled with concrete. Subsequent research on flowing superplasticizer mixes has shown definite segregation when compacted in the same manner though at a lower amplitude of vibration.

The tensile strength of the different mixes was determined by conducting the splitting tensile tests on the cylinder specimens. Only specimens stored in the air conditioned environment were subjected to splitting tensile tests. The load that was found from the splitting tensile test was converted to a tensile strength by the following equation.

$$\sigma = \frac{2\ P}{\pi\ l\ d}$$

where: σ = splitting tensile strength, psi.

P = maximum applied load, lbs.

l = length of cylinder, inches.

d = diameter of cylinder, inches.

The results of the splitting tensile tests are shown in Table 5.2 and the ratio, in percent, of the tensile strength (f_t) to the compressive strength (f'_c) is shown in Table 5.3. Complete data on splitting tensile strength can be found in Appendix D. No discernible pattern is noted except for that concerned with the consistency of the concrete that was presented in Table 5.1.

Table 5.2. Mean Splitting Tensile Strengths (f_t - psi).

Aggregate	3 Days	7 Days	28 Days	90 Days
Miami Oolite	303.10	---	383.30	469.51
Brooksville	247.84	334.23	259.51	298.86
Ft. Thompson	---	413.18	388.60	---
Calera	---	391.70	496.92	---

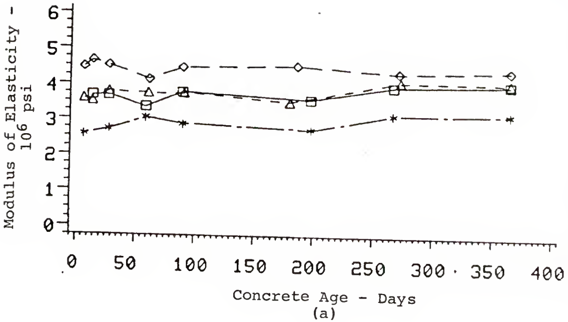
Table 5.3. Ratio (in percent) of Mean Tensile Strength to Mean Compressive Strength.

Aggregate	3 Days	7 Days	28 Days	90 Days
Miami Oolite	8.7	---	7.2	8.0
Brooksville	11.6	10.4	6.7	7.3
Ft. Thompson	---	9.0	7.3	---
Calera	---	8.6	10.3	---

Modulus of Elasticity

The modulus of elasticity is presented in Figure 5.2 and Table D.7 in Appendix D. The modulus of elasticity was determined from testing two cylinder specimens and graphical determining the mean of the two moduli. The data for the specimens stored in the air conditioned room are shown in Figure 5.2.(a). and the data for the specimens stored in the moist room are shown in Figure 5.2.(b). The modulus of elasticity follows the same pattern as the compressive strength data. For the specimens stored in the air conditioned room, Figure 5.2.(b)., the modulus of elasticity shows little if any increase with time and in the case of the Calera concrete a slight drop in the modulus with increasing time is noted. This drop in the modulus of elasticity of the Calera concrete with time parallels the decrease in its compressive strength with time and is probably caused by the same factors.

With the exception of the Calera concrete the concrete mixes that were stored in the air conditioned room, Figure 5.2.(a), do not show the rapid increase in strength during the first 60 days as was shown in the compressive strengths (Figure 5.2.(b).). The Calera concrete did show a particularly rapid increase in the modulus of elasticity up to 60 days and it continued to increase even at 365 days of age. The very sharp and inconsistent drop in the modulus



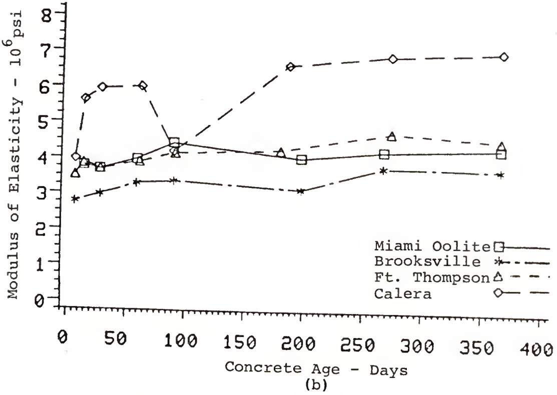


Figure 5.2. Influence of Aggregate on the Modulus of Elasticity of Concrete Stored in the (a) Air Conditioned Room and (b) the Moist Room.

of elasticity of the Calera concrete at 90 days of age is probably attributable to an instrumentation problem but the exact source is unknown.

The initial elastic strains were calculated based on the modulus of elasticity and compared to the actual elastic strains (Table D.8, Appendix D). The actual elastic strains were on the whole 24 percent greater than the predicted strain. This result is in agreement with the ACI (1) and Bazant (12) results.

Analysis Methodology for Factors Affecting Creep

The analysis of the data to consider the influence of different factors on the creep of the concrete mixtures will use the parameter known as specific creep. This parameter is found by dividing the creep by the sustained stress (in psi) applied to the specimen. The creep was found by first reducing the raw data from the measurements with the Whittemore gage to strains measured from the time of loading. These strains were found by dividing the deflection, found by subtracting the reading from the Whittemore gage at the time in question from the initial reading taken just before application of the sustained stress, by the gage length of eight inches at 70 degrees F. This strain will be referred to as the total strain in this paper, and contains the creep strain, the elastic strain and the shrinkage strain. The three strain readings from each cylinder were averaged to obtain a single strain for each cylinder. This averaging was done to eliminate some of the effects of eccentric loading if present. Typically the individual strain readings were within 20 percent of the mean.

The shrinkage strain was found in the same manner as the creep strain. The data for the individual cylinder strains were then fitted to the general equation from ACI Committee 209 (1) for shrinkage:

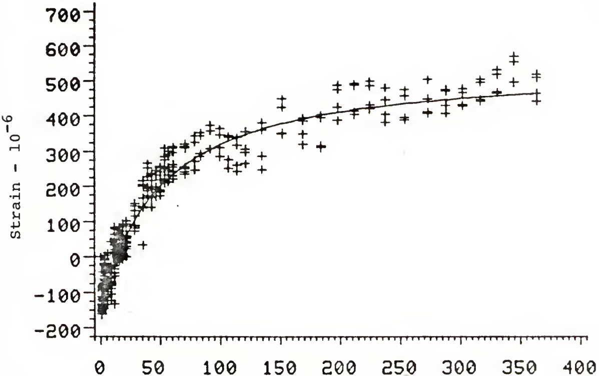
$$Y = (A \cdot X^B)/(C + X^B) + D$$

where: Y = strain at time X (inches/inch)
X = time in days
A = factor evaluated by curve fitting
B = factor evaluated by curve fitting
C = factor evaluated by curve fitting
D = non-zero intercept for Miami Oolite data only.

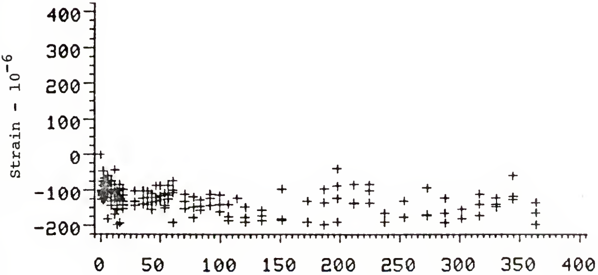
The equation was fitted to the shrinkage strain data sets for specimens from the air conditioned room by the use of the DUD nonlinear curve fitting program in the SAS program set (45). The DUD method is the multi-variant secant method and is similar to Gauss-Newton but estimates of derivatives are done analytically. Upon examining the plot of the Miami Oolite shrinkage strain versus time for specimens stored in the air conditioned room (Figure 5.3) it became obvious that another factor "D" would have to be subtracted from equation 5.1 in order to account for the apparent initial swelling. The factors, A,B,C, and D along with the R^2 value which is an indication of the adequacy of fit of the data to the equation is given in Table 5.4. The coefficient of multiple determination (50), R^2 in Table 5.4, indicates what proportion of the total variation in the creep strain response is explained by the fitted model. An R^2 value of 1.0 is a perfect fit for an equation to a set of data, and thus the above factors when used in the appropriate equation give an excellent curve fit. An R^2 greater than 0.85 was considered a good curve fit (50). The above factors were used in equation 5.2 at the time in question in order to obtain the shrinkage to subtract from the total strain. Since the beginning of shrinkage strain measurement did not necessarily coincide with the application of the sustained stress, a correction for the shrinkage strain for the time of initial application of the sustained stress must also be subtracted (Figure 2.2). The plot of the shrinkage versus the time since the beginning of taking measurements for the specimens from the air conditioned room is shown in Figure 5.3.(a). (Miami Oolite concrete), Figure 5.4.(a).

Table 5.4. SAS Fitted Equation Factors.

Concrete	A	B	C	D	R^2
Miami Oolite	658.67	1.08	68.24	122.71	0.967
Brooksville	566.09	1.41	133.44	---	0.992
Ft. Thompson	657.63	1.01	68.30	---	0.990
Calera	470.40	0.91	25.57	---	0.929

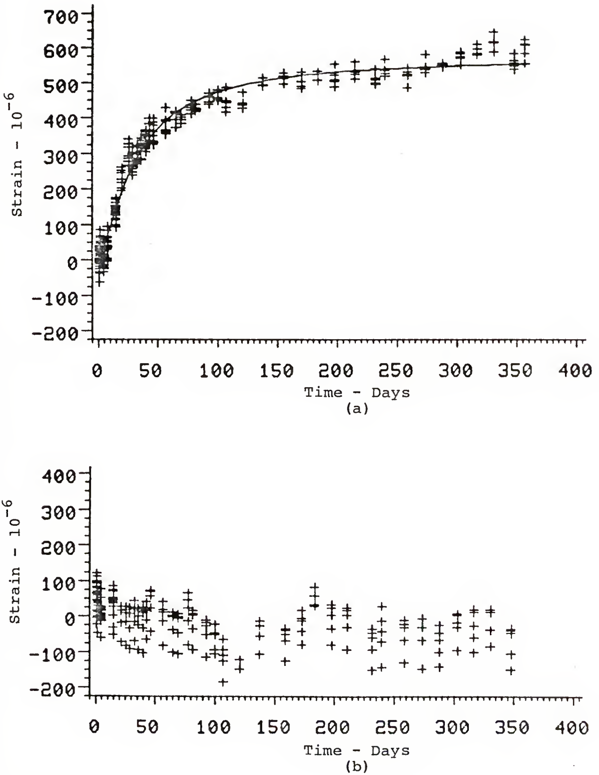


Time - Days
(a)



Time - Days
(b)

Figure 5.3. Shrinkage Strain of Miami Oolite Concrete for (a) Air Conditioned Storage and (b) Moist Room Storage (Strain Data Begins at a Concrete Age of 3 days).



(a)

(b)

Figure 5.4. Shrinkage Strain of Brooksville Concrete for (a) Air Conditioned Storage and (b) Moist Room Storage (Strain Data Begins at a Concrete Age of 3 Days).

(Brooksville concrete), Figure 5.5(a). (Ft. Thompson concrete), and Figure 5.6.(a) (Calera concrete). Along with the data set is the curve from equation 5.2 with the appropriate factors.

The plot of the shrinkage strain versus the time since measurements were begun for moist room storage for the Brooksville concrete (Figure 5.4.(b).), the Ft. Thompson concrete (Figure 5.5.(b).), and the Calera concrete (Figure 5.6.(b).) show no appreciable shrinkage for the period that measurements were taken. Thus the shrinkage was taken as zero for the specimens from the moist room for these concretes. The Miami Oolite concrete (Figure 5.3.(b).) showed a definite swelling and thus the same correction factor "D" that was used in equation 5.2 for the air conditioned specimens, a minus $122.71\ 10^{-6}$ inches/inch, was used as the shrinkage strain throughout the period of this study.

The specific creep was found by subtracting the initial reading just after the application of the sustained load from each successive strain. Since the reading after the application of the sustained load was made within 15 minutes of the application of the load it is being considered as the initial elastic strain. What remains after subtracting the initial elastic strain is the creep strain. The creep strain is then divided by the applied sustained stress to obtain the specific creep..

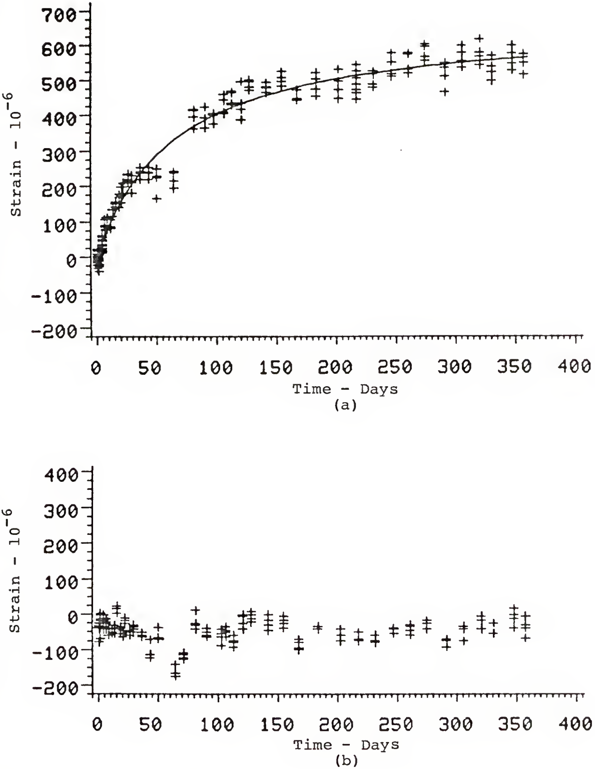
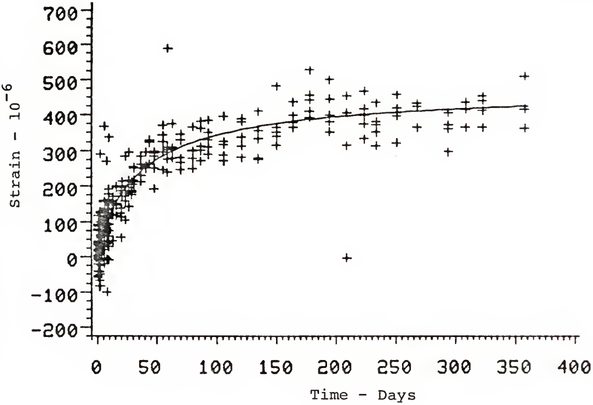
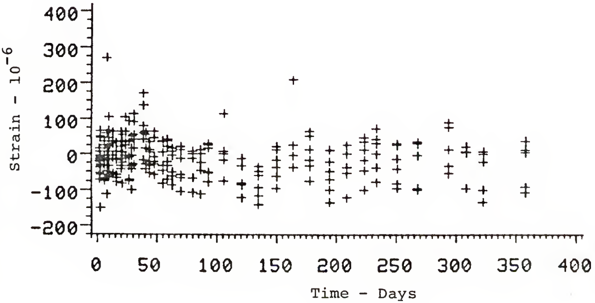


Figure 5.5. Shrinkage Strain of Ft. Thompson Concrete for (a) Air Conditioned Storage and (b) Moist Room Storage (Strain Data Begins at a Concrete Age of 7 Days).



(a)



(b)

Figure 5.6. Shrinkage Strain of Calera Concrete for (a) Air Conditioned Storage and (b) Moist Room Storage (Strain Data Begin at a Concrete Age of 7 Days).

Influence of Age of Loading

Miami Oolite Concrete

The plot of the specific creep versus the concrete age for loads of 50% of the compressive strength is shown in Figure 5.7. The data for each cylinder (4 test cylinders per test frame) are shown for each of the three times of loading (3, 28 and 90 days) for specimens stored in the air conditioned room. The specimens loaded at a concrete age of three days show a higher specific creep than the specimens loaded at 28 and 90 days. The specimens loaded at 28 and 90 days have approximately the same specific creep at the end of the testing period of 365 days. These results are consistent with the result found by Neville (39) and by Timusk and Ghosh (46); that the creep of concrete is generally not influenced by the age of the concrete after most of the hydration of the cement has taken place after about 28 days.

Brooksville Concrete

Figure 5.8 shows the effect of the age of loading on the Brooksville concrete. This plot shows 50 percent of the compressive strength (f_c') loadings for ages of 7, 28 and 90 days. Two frames were loaded at 28 days to a compressive strength of 50 percent f_c' so that four sets of data are shown. The specific creep at a loading of seven days is, like the Miami Oolite, higher than the specific creep for the 28 and 90 days loadings. Both sets of data for the 28 days

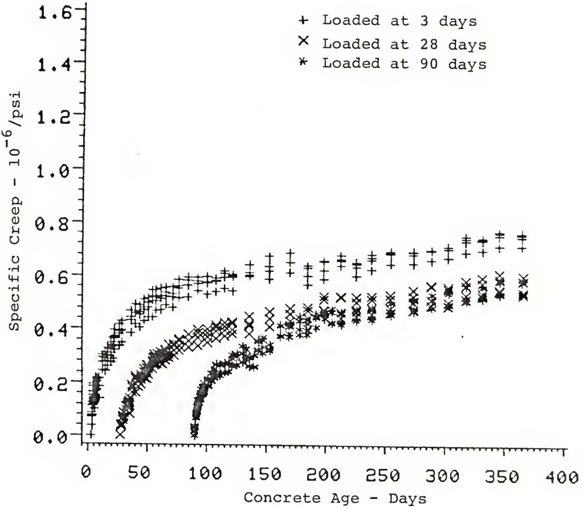


Figure 5.7. Influence of Age of Loading on Creep of Miami Oolite Concrete Loaded to 50% f'_c in, Air Conditioned Storage.

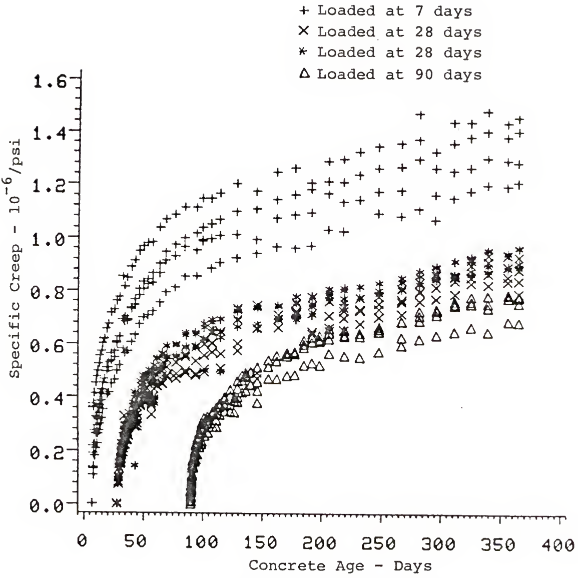


Figure 5.8. Influence of Age of Loading on Creep of Brooksville Concrete Loaded to 50% f'_c in Air Conditioned Storage.

loading do approximately coincide. The data from the 90 days loading is approaching the 28 days loading specific creep at a concrete age of 365 days. At 365 days the higher specific creep data from the 90 days loading are coinciding with the lower specific data from the 28 days loading. There is no indication that the 28 day curve and the 90 day curve will converge at any future time. The difference between the 7 day specific creep data and the 28 day specific creep is approximately three times the difference between the 28 day data and the 90 day data.

Ft. Thompson Concrete

The Ft. Thompson concrete specimens stored in the air conditioned room were loaded at 7 and 28 days to 50 percent of their compressive strength and a plot of the data is shown in Figure 5.9. The specific creep data quickly converges and shows that the Ft. Thompson concrete mix is less sensitive with respect to creep to the age at loading than either the Miami Oolite, Brooksville or the Calera (to be discussed next) concretes.

Calera Concrete

Figure 5.10 shows the specific creep data for loadings of 7 and 28 days for the Calera concrete. The specimens were all from the air conditioned room and all were loaded to 50 percent of the compressive strength. The Calera concrete shows a definite difference between the creep responses

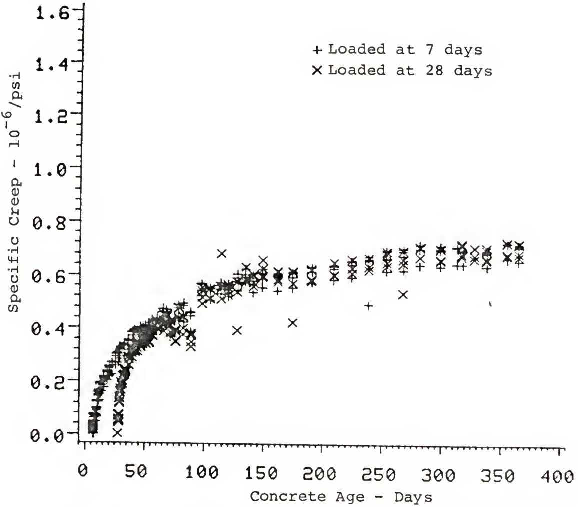


Figure 5.9. Influence of Age of Loading on Creep of Ft. Thompson Concrete Loaded to 50% f'_c in Air Conditioned Storage.

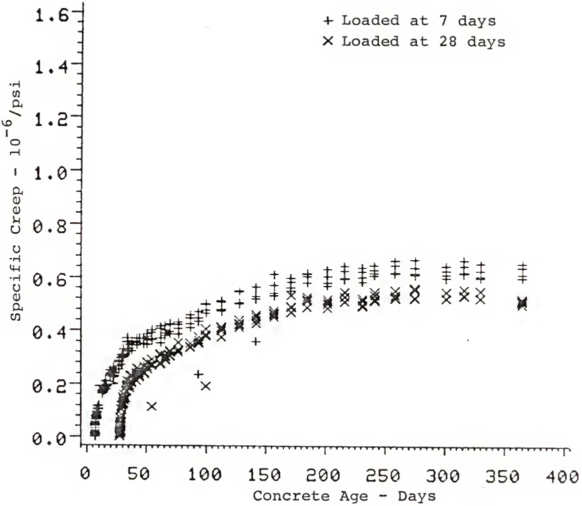


Figure 5.10. Influence of Age of Loading on Creep of Calera Concrete Loaded to 50% f'_c in Air Conditioned Storage.

between the 7 and 28 day loadings. Thus, for three out of four of the concretes tested the age of loading appeared to have some effect, up to a concrete age of 28 days, on the creep response of the concrete. Beyond 28 days there is only data from the Miami Oolite and Brooksville concretes and for these two concretes the age of the concrete loaded after 28 days does not appear to appreciably affect the creep response. The exception is the Ft. Thompson concrete which did not appear sensitive to the age of loading.

Influence of Relative Humidity

Miami Oolite Concrete

The Miami Oolite concrete was tested under the two different storage conditions at a concrete age of 28 days and an applied stress of 50 percent of the compressive strength. The data for both the specimens stored in the moist room and the air conditioned room are shown in Figure 5.11. At 365 days the creep response for the specimens in the air conditioned room is approximately 2.5 times that for specimens stored in the moist room. The rate of creep for the specimens from the moist room is also less than that of the specimens from the air conditioned room at a concrete age of 365 days.

Brooksville Concrete

The Brooksville concrete was tested in both the moist room and the air conditioned room at an applied stress of 50 percent of the compressive strength at a concrete age of 28 days.

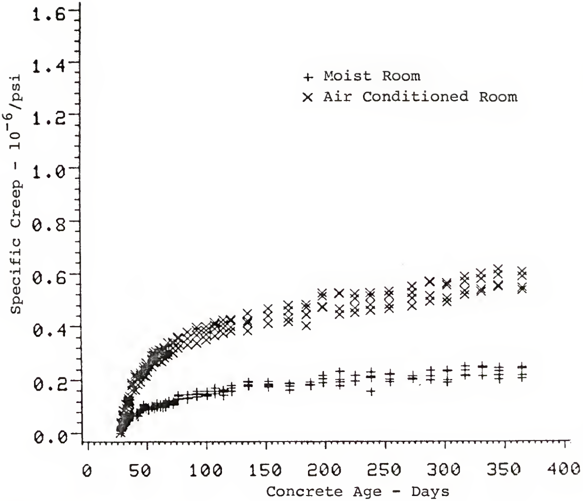


Figure 5.11. Influence of Relative Humidity on Creep of Miami Oolite Concrete Loaded at 28 Days to 50% f'_c.

Each storage condition had two frames loaded to an applied stress of 50 percent of the compressive stress. The plot of these four sets of data is shown in Figure 5.12. The specific creep of the air conditioned room specimens is approximately 3.8 times that of the moist room specimens at a concrete age of 365 days. The rate of creep is also greater for the air conditioned stored specimens than for the moist room stored specimens.

Ft. Thompson Concrete

The Ft. Thompson concrete was tested at an applied load of 50 percent of the compressive strength at a concrete age of 28 days with specimens in the air conditioned room and the moist room. The results are plotted in Figure 5.13. The specific creep of the air conditioned specimens is approximately 3.4 times the specific creep for specimens stored in the moist room. The specimens stored in the moist room showed little if any increase in creep with time, whereas the specimens stored in the air conditioned room showed a definite increase, although at a declining rate, in creep with increasing age of the concrete.

Calera Concrete

The Calera aggregate was also tested at an applied load of 50 percent of the compressive strength at a concrete age of 28 days with specimens in the air conditioned room and moist room. The results are plotted in Figure 5.14. The

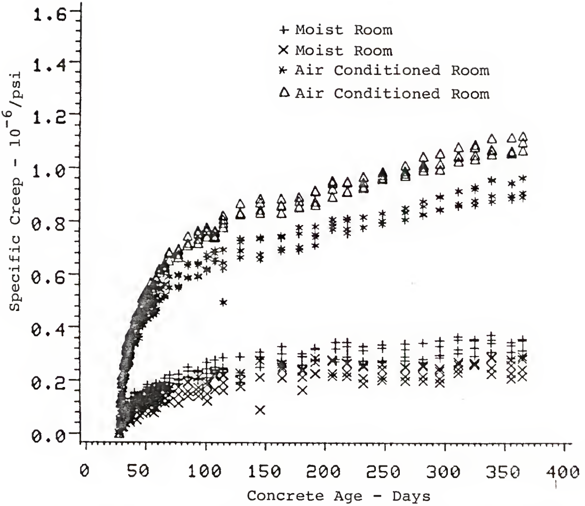


Figure 5.12. Influence of Relative Humidity on Creep of Brooksville Concrete Loaded at 28 Days to 50% f'_c.

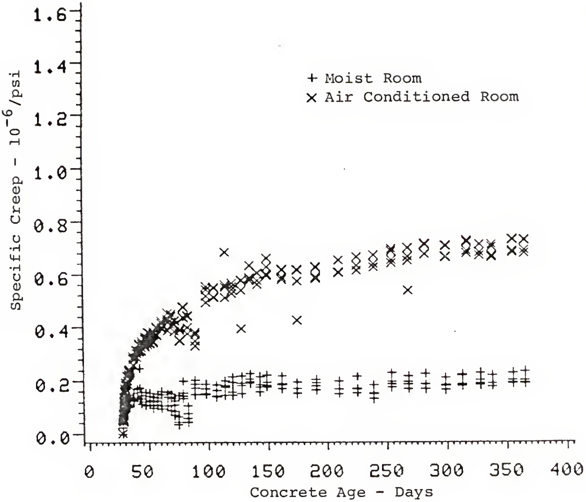


Figure 5.13. Influence of Relative Humidity on Creep of Ft. Thompson Concrete Loaded at 28 Days to 50% f'_c.

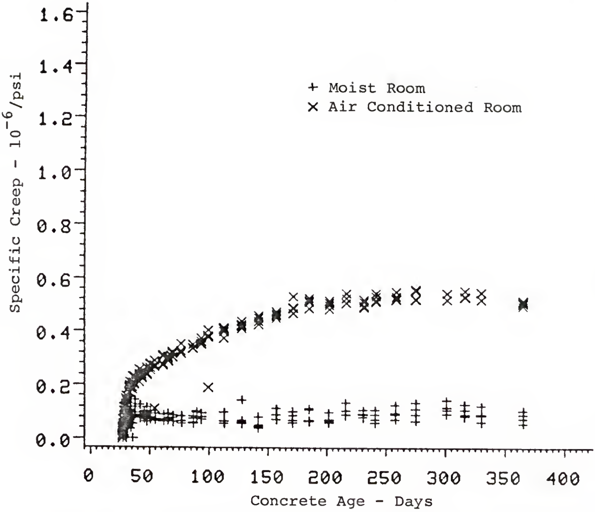


Figure 5.14. Influence of Relative Humidity on Creep of Calera Concrete Loaded at 28 Days to 50% f'_c.

plot of the Calera concrete is very similar to Figure 5.13 of the Ft. Thompson concrete. The specific creep of the air conditioned specimens is approximately 3.2 times that of the moist room specimens at a concrete age of 365 days. The specimens from the moist room appear to have experienced very little creep after the first 10 days of being under a sustained load. The specimens from the air conditioned room still experienced creep, though at a declining rate, at 365 days.

Influence of the Level of Applied Stress

In the analysis of creep the most common assumption is that the rate of creep in linear up to an applied stress of about 50 percent of the compressive strength. Neville (39) makes this assumption and research has proven its value. The use of the specific creep for comparing creep responses is based on this assumption. In this section this assumption will be examined by comparing the specific creep response curves for different levels of applied stress at the same concrete age at application.

Miami Oolite Concrete

Figure 5.15 compares the specific creep response curves for applied stresses of 25 percent and 50 percent of the compressive strength of the Miami Oolite concrete. The specimens were from the air conditioned room and the sustained stress was applied at 28 days. If the creep response was perfectly linear, the two sets of data curve would coincide. In this

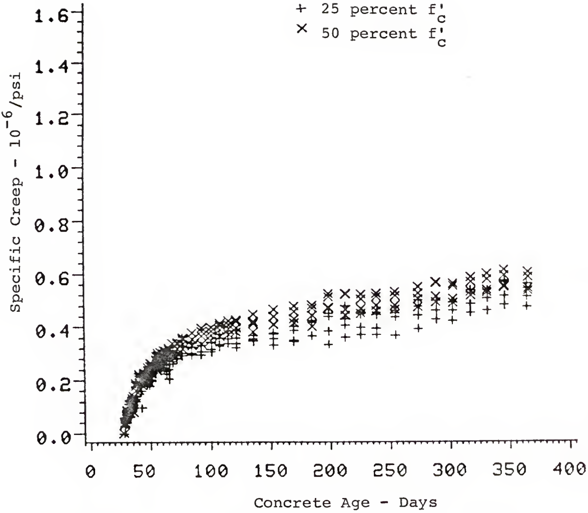


Figure 5.15. Influence of Level of Applied Stress on Creep of Miami Oolite Concrete in Air Conditioned Storage and Loaded at an Age of 28 days.

case they almost coincide. The specimens with the higher creep response from the 25 percent stress level coincide with the lower creep responses from specimens at the 50 percent stress level.

Brooksville Concrete

The Brooksville concrete had two levels of sustained stress applied (25 and 50 percent) for specimens in both the air conditioned room and the moist room. The 50 percent stress level was duplicated in both environments to give a total of three data sets per environment. All specimens were loaded to their applied stress at 28 days.

Air conditioned storage

The data for the specimens tested in the air conditioned room are shown in Figure 5.16. The data from the specimens at the 25 percent stress level coincide with data from one of the sets of data from specimens at the 50 percent stress level but not with the other. The set of data for the other 50 percent stress level is approximately 12 percent higher than the average of the other two data sets. This indicates that the creep responses at the 50 percent stress level have a variability of about 10 percent. More research would be warranted before assuming that all the data sets could have a similar variability.

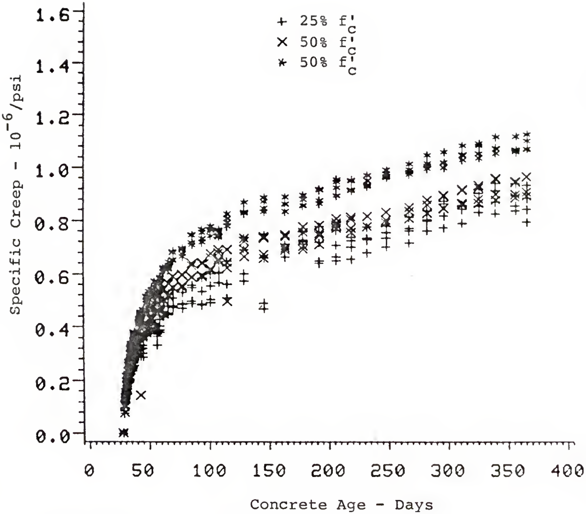


Figure 5.16. Influence of Level of Applied Stress on Creep of Brooksville Concrete in Air Conditioned Storage and Loaded at an Age of 28 Days.

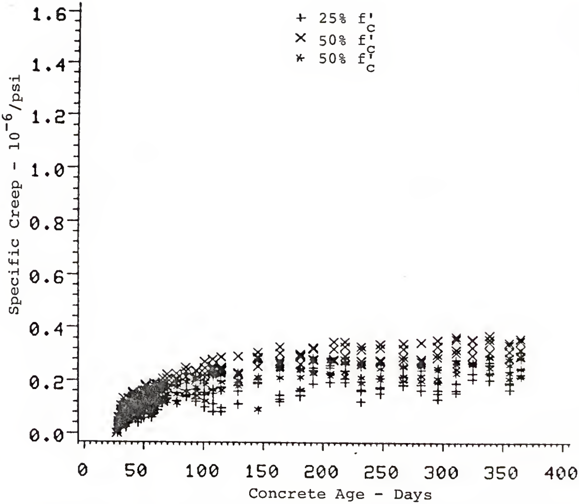


Figure 5.17. Influence of Level of Applied Stress on Creep of Brooksville Concrete in Moist Room Storage and Loaded at 28 Days.

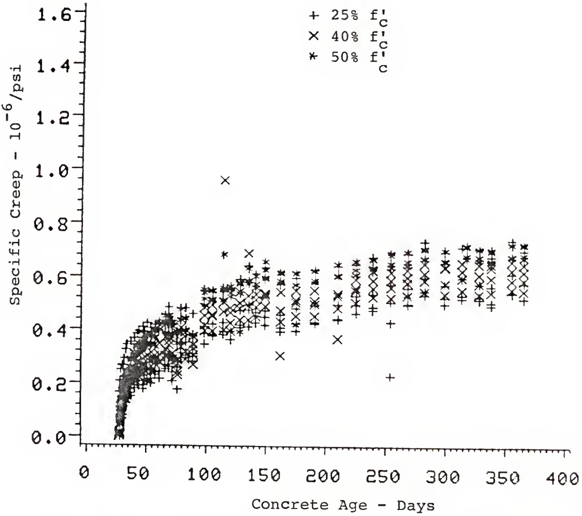


Figure 5.18. Influence of Level of Applied Stress on Creep of Ft. Thompson Concrete in Air Conditioned Storage and Loaded at 28 Days.

Moist room storage

The data for the specimens tested in the moist room are shown in Figure 5.17. The data do not show the divergence that was shown in the plot of air conditioned room specimens (Figure 5.16). The 25 percent applied stress curve and one of the 50 percent applied stress curves coincide very well. The other 50 percent applied stress data set coincides with only the higher creep responses from the other 50 percent applied stress and the 25 percent applied stress.

Ft. Thompson Concrete

The Ft. Thompson concrete was tested at three sustained stress levels (25, 40 and 50 percent) at a concrete age of 28 days. The results are shown in Figure 5.18. The curves do coincide to some degree but it should be noted that the higher the stress level the higher the specific creep. The 50 percent stress level curve is slightly higher than the 40 percent stress level curve and is in turn slightly higher than the 25 percent stress level curve. Each stress level data set coincides with the lower creep responses of the next higher level. The 25 percent stress level does not coincide with the 50 percent stress level creep responses.

Calera Concrete

The Calera concrete was subjected to four different stress levels (25, 33, 40 and 50 percent of the compressive strength). All specimens were from the air conditioned room and all had the sustained stress applied at 28 days. The plot of this data is shown in Figure 5.19. All data sets coincide very well and the assumption of linear of creep with applied stress level is very good for this concrete.

Influence of Different Aggregates

The analysis of the influence of different aggregates upon the creep and shrinkage of concretes made with them is one of the major objectives of this project. Each of the aggregates was subjected to the effects of age of loading, storage conditions and different levels of applied stress. All other factors that were known to influence creep response of concrete was kept constant of minimized. The influence of the aggregate will be analyzed by examining specific creep versus concrete age plots at different ages at loading, at different applied stress levels and at different storage environments.

Creep Response to Different Aggregates

Stress applied at 7 days

For the case where a sustained stress was applied at a concrete age of 7 days (3 days in the case of Miami Oolite concrete) only specimens from the air conditioned room were tested. Figure 5.20 shows each of the data sets for each aggregate for an applied stress of 50 percent of the compressive stress. The curves for the Miami Oolite, Ft. Thompson and Calera concretes coincide very well. The curve for the

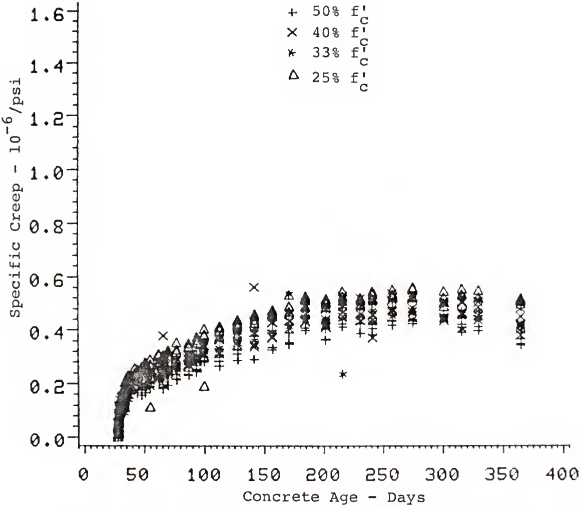


Figure 5.19. Influence of Level of Applied Stress on Creep of Calera Concrete in Air Conditioned Storage and Loaded at an Age of 28 Days.

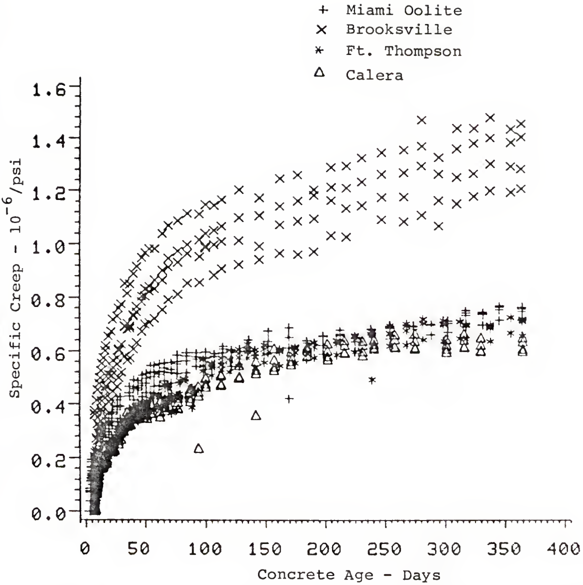


Figure 5.20. Influence of Aggregates on Creep of Concretes in Air Conditioned Storage and Loaded at a Concrete Age of 7 Days to 50 Percent f'_c.

Brooksville concrete has a much higher specific creep response than the other concretes (about 1.7 times). The dispersion among each individual cylinder is also very high. It should be noted that the Brooksville concrete also had the highest slump, the lowest compressive and tensile strengths, and the lowest modulus of elasticity of the four concretes.

Stress applied at 28 days

In the case where the sustained stress was applied at 28 days, the testing of specimens was further broken down into those tested in the moist room or the air conditioned room. The air conditioned room stored specimens were further tested at two stress levels, 25 and 50 percent of the compressive strength.

Air conditioned storage at 25 percent stress level. Figure 5.21 is a plot of the data sets of the specimens from the four aggregate concretes that were tested in the air conditioned room to a stress level of 25 percent of the compressive strength. Though not as severe, this figure shows the same pattern as that for Figure 5.20. Namely that the Brooksville concrete is showing a significantly higher creep response (approximately 1.7 times) than the mean of the other three aggregates. One of the Ft. Thompson concrete cylinders has a stress response almost as high as the Brooksville concrete.

Air conditioned storage at 50 percent stress level. Figure 5.22 is a plot of the data sets of the specimens from

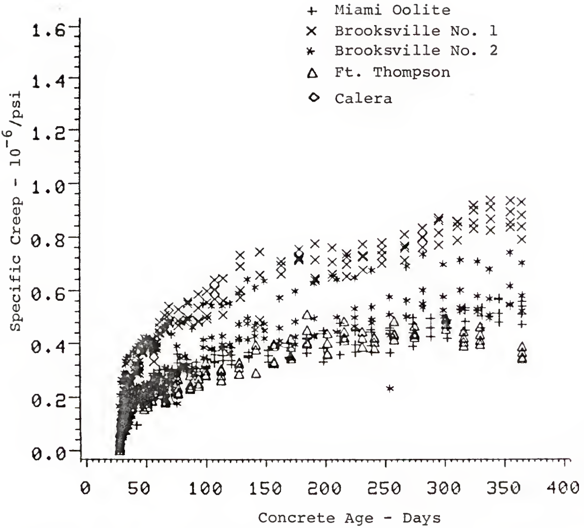


Figure 5.21. Influence of Aggregate on Creep of Concretes in Air Conditioned Storage and Loaded at a Concrete Age of 28 Days to 25 Percent f'_c.

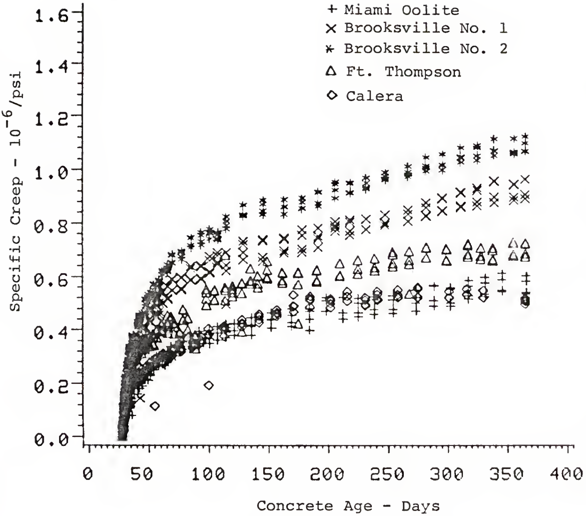


Figure 5.22. Influence of Aggregates on Creep of Concretes in Air Conditioned Storage and Loaded at an Age of 28 Days to 50 Percent f'_c.

the four aggregate concretes (two data sets for the Brooksville concrete) that were tested in the air conditioned room at a sustained stress level of 50 percent of the compressive strength. The two data sets from the Brooksville concrete have a higher creep response than the other three aggregate concretes. The Miami Oolite concrete and the Calera concrete have very similar creep response curves. The creep response curve for the Ft. Thompson concrete is approximately 12 percent higher than that for the combined Miami Oolite and Calera concrete response curves at a concrete age of 365 days.

Moist room storage at 50 percent stress level. Figure 5.23 is a plot of the data sets of the specimens from the four aggregate concretes (two data sets for the Brooksville concrete) that were tested in the moist room to a sustained stress level of 50 percent of the compressive stress. The creep responses for all the concretes are similar. One of the two Brooksville concrete data sets does appear to be slightly higher than the other data sets but it coincides with them at its lower specific creep responses.

Stress applied at 90 days

For the case where the sustained stress was applied at a concrete age of 90 days only specimens from the air conditioned room were tested. Figure 5.24 shows the creep response curves for the Miami Oolite and Brooksville concretes which were the only ones tested at 90 days. The creep response

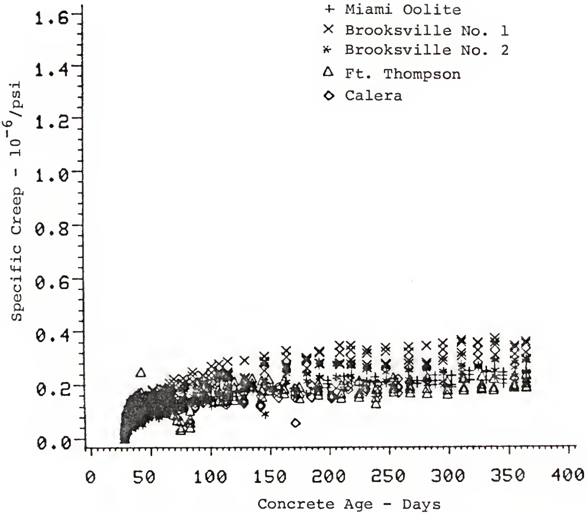


Figure 5.23. Influence of Aggregates on Creep of Concretes in Moist Room Storage and Loaded at an Age of 28 Days to 50 Percent f'_c.

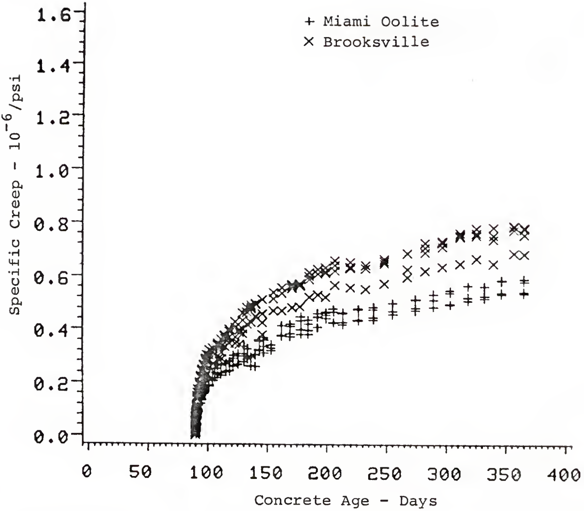


Figure 5.24. Influence of Aggregates on Creep of Concretes in Air Conditioned Storage and Loaded at an Age of 90 Days to 50 Percent f'_c.

curve of Miami Oolite does not coincide with the Brooksville data. The specific creep responses of the latter being approximately 1.3 times greater.

Shrinkage Response to Different Aggregates

The shrinkage response was investigated earlier in this chapter in order to determine a shrinkage strain for subtraction from the total strain. The difference between the shrinkage strains for the four aggregates will now be examined in greater detail.

Shrinkage in the moist room

As was reported previously, the Miami Oolite concrete showed some initial swelling whereas the other concretes (Brooksville, Ft. Thompson and Calera) showed no appreciable shrinkage or swelling during the period of measurements.

Shrinkage in the air conditioned room

The shrinkage for the specimens stored in the air conditioned room versus time of measurement is plotted in Figure 5.3.(a). for Miami Oolite concrete, Figure 5.4.(a). for Brooksville concrete, Figure 5.5.(a). for Ft. Thompson concrete and Figure 5.6.(a). for Calera concrete. The shrinkage strain for the Miami Oolite concrete and the Calera concrete are very close at the end of 365 days. Likewise the Ft. Thompson and the Brooksville concretes have almost identical shrinkage strains after 365 days. The Ft. Thompson and

Brooksville have shrinkage strains approximately 20 percent greater than those of the Miami Oolite and Calera aggregates.

Indirect Tensile Creep

The data from the indirect tensile creep were not sufficient due to data scatter to make a comparison with the compressive creep results. The indirect tensile creep plots are presented in Appendix A.

Creep Recovery

Five of the testing frames were unloaded at a concrete age of 365 days. Two of the frames contained Miami Oolite concrete and three of the frames contained Brooksville concrete. The Miami Oolite concretes were both from the air conditioned room and one had been loaded at a concrete age of 3 days and the other at a concrete age of 90 days. A plot of the elastic strain plus the creep strain versus the time since loading are shown in Figure 5.25 for the Miami Oolite stored in the moist room and loaded at three days, Figure 5.26 for Miami Oolite stored in the moist room and loaded at 90 days, Figure 5.27 for the Brooksville stored in the moist room and loaded at 28 days, Figure 5.28 for the Brooksville stored in the air conditioned room and loaded at 28 days, and Figure 5.29 for the Brooksville stored in the air conditioned room and loaded at 90 days.

Table 5.5 summarizes the results of the unloading tests. A comparison of the instantaneous recovery strain to the

Table 5.5. Creep Recovery.

Aggregate	Age at Loading (Days)	Storage Environment *	Strain on Loading ε_c	Max. Strain	Instantaneous (Strain) Recovery ε_r	Creep Recovery (Strain)	Permanent Set (Strain)
Miami Oolite	3	A/C	791	2080	500	46	1534
Miami Oolite	90	A/C	1158	2796	917	196	1683
Brooksville	28	M/R	709	1246	484	152	610
Brooksville	28	A/C	985	3091	685	192	2217
Brooksville	90	A/C	968	2496	679	210	1608

*A/C: Air conditioned environment

M/R: Moist room environment

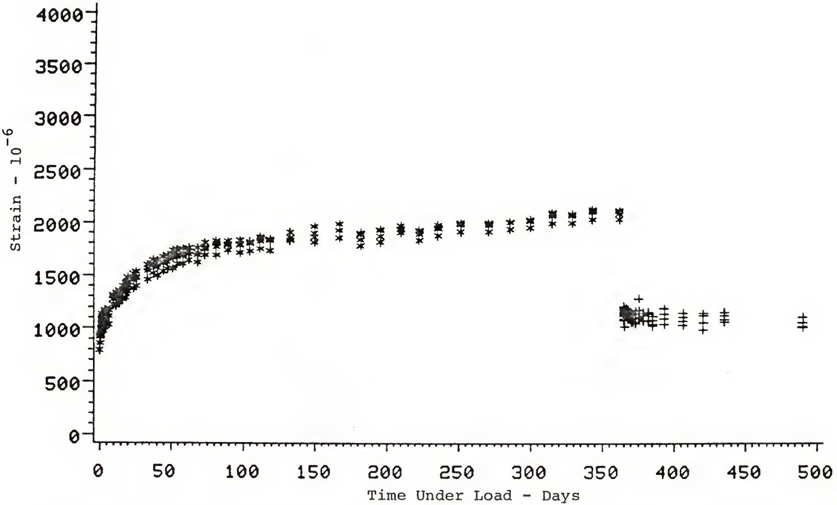


Figure 5.25. Creep and Creep Recovery of Miami Oolite Concrete Stored in the Air Conditioned Environment and Loaded at 3 Days.

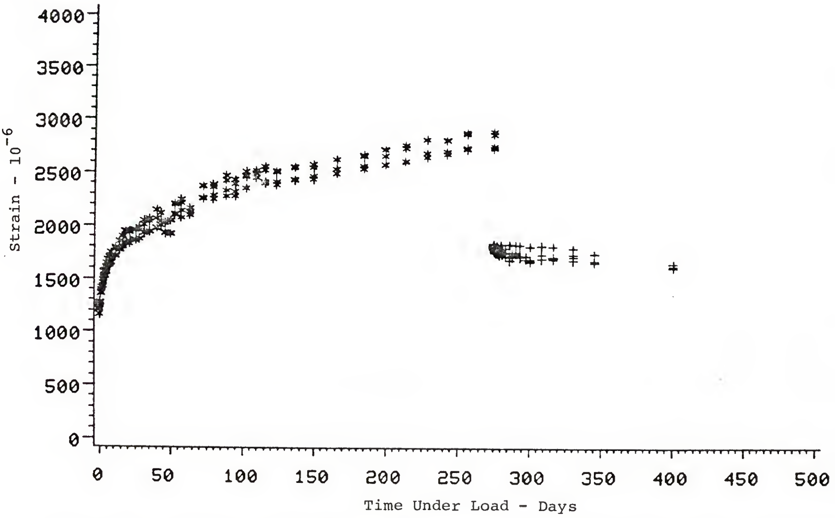


Figure 5.26. Creep and Creep Recovery of Miami Oolite Concrete Stored in the Air Conditioned Environment and Loaded at 90 Days.

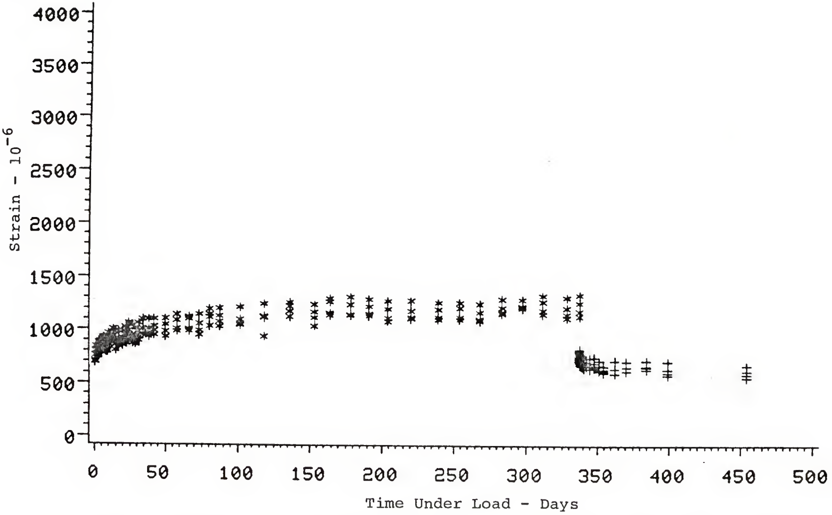


Figure 5.27. Creep and Creep Recovery of Brooksville Concrete Stored in the Moist Room Environment and Loaded at 28 Days.

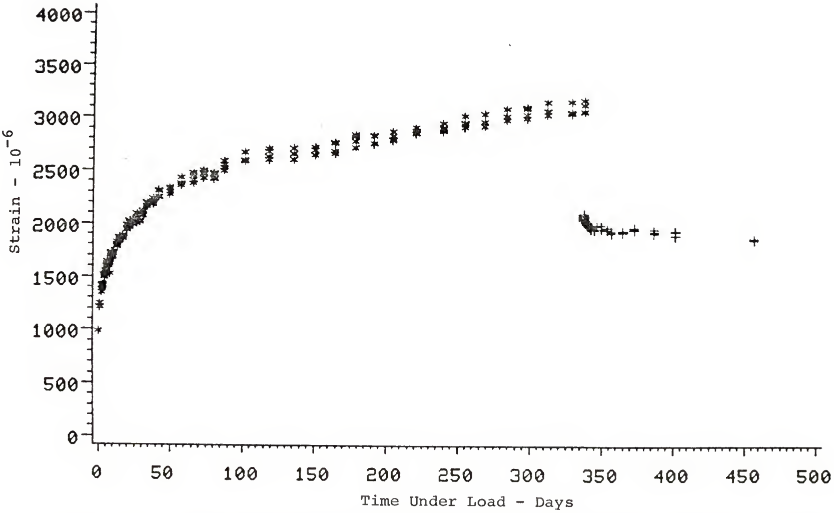


Figure 5.28. Creep and Creep Recovery of Brooksville Concrete Stored in the Air Conditioned Environment and Loaded at 28 Days.

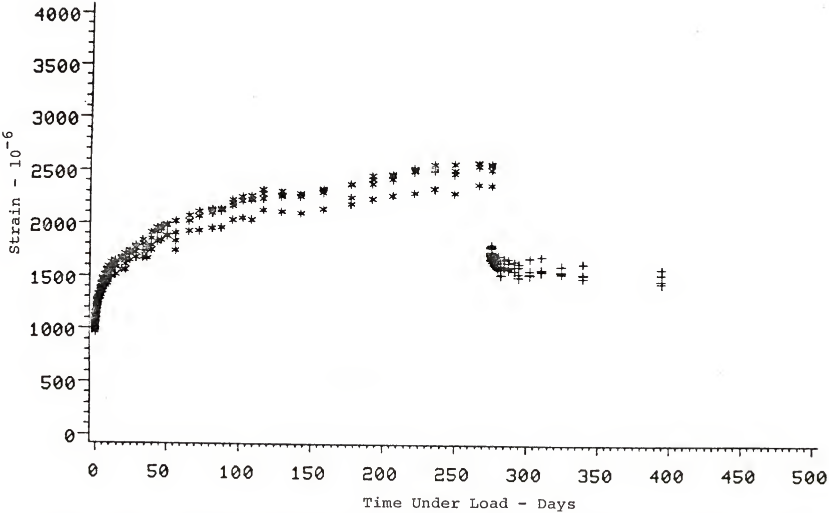


Figure 5.29. Creep and Creep Recovery of Brooksville Concrete Stored in the Air Conditioned Environment and Loaded at 90 Days.

initial strain on loading is shown in Table 5.6. A comparison of Tables 5.5 and 5.6 with Figure 2.3 shows that, due to the instantaneous recovery not equalling the elastic strain, the curves do not fit the Burger's Model that require this equivalence. There is a substantial difference between the instantaneous recovery strain and the initial strain on loading for the Miami Oolite concrete at 3 day and 90 day loading. This difference is probably due to the early (3 day) loading of the Miami Oolite while still much hydration of the cement paste was in progress. A further indication of this is that the Brooksville concretes show no significant difference in this ratio regardless of storage environment or age of loading (28 and 90 days).

Table 5.6. Ratio of Initial Strain to Instantaneous Recovery.

Aggregate	Age at Loading	Storage	$(\varepsilon_r/\varepsilon_e) \times 100$*
Miami Oolite	2	A/C	63.21
Miami Oolite	90	A/C	79.19
Brooksville	28	M/R	68.27
Brooksville	28	A/C	69.54
Brooksville	90	A/C	70.14

* ε_r = Instantaneous Recovery

ε_e = Initial Strain on Loading

CHAPTER VI
CREEP AND SHRINKAGE PREDICTION MODELS

Three prediction models will be investigated in this chapter to see how they fit the experimental data. The models that will be examined are the ACI Committee 209 model (1), the CEB-FIP (25) model and the simplified BP model (17). The ultimate creep coefficient used in the ACI model will be examined for possible modification to improve the model for use with concretes made with Florida aggregates. In addition, the Burgers model was fitted to the data for evaluation of its suitability for prediction of creep.

Comparison of Creep Models with Experimental Data

The primary assumption in using the three prediction methods was that nothing was known about the concretes except their 28 day compressive strengths. The ACI model has the allowance of varying the ultimate creep coefficient used in the method and the BP method has a more elaborate method that can make use of short term creep data. The CEB method does not have any direct method for modification. The other assumption is that the air conditioned storage environment was a constant 60 percent relative humidity. The air conditioned storage was not a constant humidity and varied from 40 percent relative humidity in the winter to 80 percent relative humidity in the summer.

The ACI model and the CEB model give their results in the form of a creep coefficient, which is the ratio of the creep deformation to the elastic deformation. The BP model gave its results in the form of the specific creep and in order to have comparable comparisons of the prediction models the specific creep was converted to a creep coefficient by the method described by Bazant and Panula (17). The ACT model and the BP model gave results in the form of an equation whereas the CEB model gave a creep coefficient at a specified time after loading. The CEB model was evaluated at 10, 20, 40, 60, 100, 200, 300 and 365 days after the application of the load. A continuous curve was fitted to the discrete data points by use of a cubic spline (45). For the sake of consistency all prediction models had predictions to 365 days after the application of the load, even though the experimental data did not extend that far. The coefficients used in the three models are provided in Appendix D.

Age of Loading at 3/7 Days

The comparison of the three creep prediction models are shown in Figures 6.1, 6.2, 6.3, and 6.4 for the Miami Oolite concrete, the Brooksville concrete, the Ft. Thompson concrete, and the Calera concrete, respectively. The fit of the prediction models to the Miami Oolite concrete that was

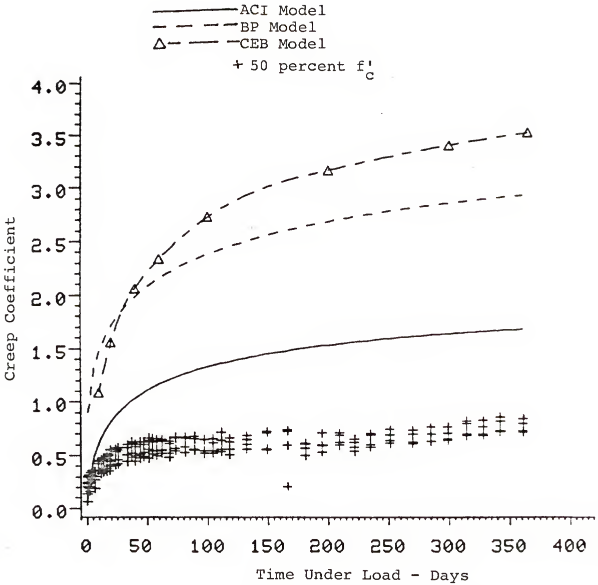


Figure 6.1. Comparison of Creep Prediction Models with Miami Oolite Concrete Stored in the Air Conditioned Room and Loaded at 3 Days to 50 Percent of f'_c.

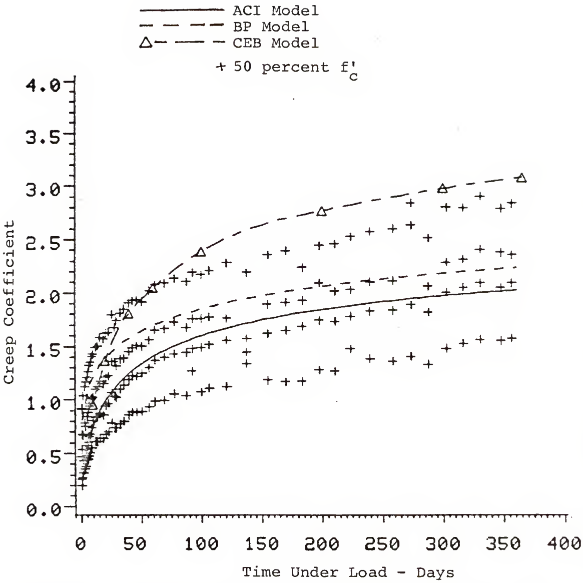


Figure 6.2. Comparison of Creep Prediction Models with Brooksville Concrete Stored in the Air Conditioned Room and Loaded at 7 Days to 50 Percent f'_c.

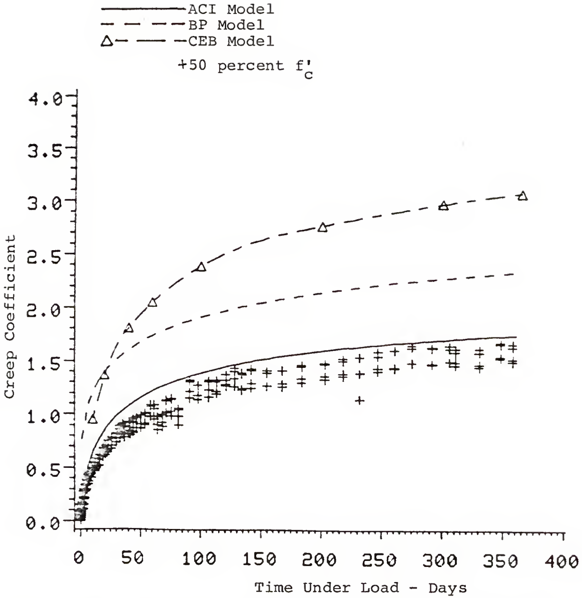


Figure 6.3. Comparison of Creep Prediction Models with Ft. Thompson Concrete Stored in the Air Conditioned Room and Loaded at 7 Days to 50 Percent of f'_c.

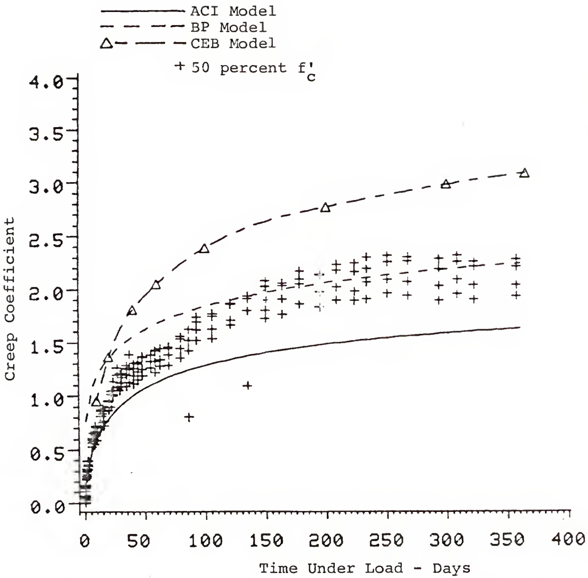


Figure 6.4. Comparison of Creep Prediction Models with Calera Concrete Stored in the Air Conditioned Room and Loaded at 7 Days to 50 Percent of f'_c.

loaded at 3 days is particularly poor. The ACI model predictions are 104 percent greater, the BP model predictions are 255 percent, and the CEB model predictions are 320 percent greater than the creep coefficient that was actually observed in the experimental data at the end of the testing period. The scatter of the Brooksville concrete that was loaded at 7 days is so large as to make comparisons difficult. The ACI model and the BP model predictions do fall within the scatter of data whereas the CEB model predictions fall above the experimental data. In Figure 6.3, Ft. Thompson concrete loaded at 7 days, the ACI model predictions fall just above the experimental data whereas the BP model predictions are 1.5 times and the CEB model predictions are 1.9 times the average experimental creep coefficient at 365 days. In the case of the Calera concrete (Figure 6.4) the BP model predictions fall within the experimental data scatter whereas the ACI model predictions are 23 percent lower and the CEB model predictions are 40 percent higher than the experimental average creep coefficient at the end of the testing period. The ACI model had the best predictions or tied in 3 out of 4 of the concretes (Miami Oolite, Brooksville, and Ft. Thompson).

Age of Loading of 28 Days

Air conditioned storage

The comparison of the creep prediction models with the concretes loaded at 28 days and stored in the air conditioned room are shown in Figures 6.5, 6.6, 6.7, and 6.8. In Figure 6.5 the ACI model and the CEB model predictions for Miami Oolite concrete are almost identical. Both models predict creep coefficients that are 47 percent higher than the average experimental results at the end of the test period. The BP model predictions are 72 percent higher for the same period. The fit of the model predictions is better for the Brooksville concrete (Figure 6.6). The BP model predictions are the best and are about 16 percent greater than the average experimental creep coefficients at the end of the testing period. The CEB model is next best at 28 percent greater than the experimental results. The ACI model is the worst at 38 percent greater than the average experimental creep coefficient at the end of the test period. The Ft. Thompson concrete stored in the air conditioned room and loaded at 28 days (Figure 6.7) showed considerable scatter in the data due to the effects of one cylinder from the 25 percent f'_c results. If this cylinder is ignored, the fit of all the prediction models is very good and they all parallel the high side of the experimental creep coefficients. The ACI model predictions are the best at 18 percent greater than the average experimental creep coefficients. The BP model predictions are very close to the ACI model predictions and are 22 percent higher than the average of the experimental creep coefficients. The CEB model predictions are 28 percent greater than the experimental results.

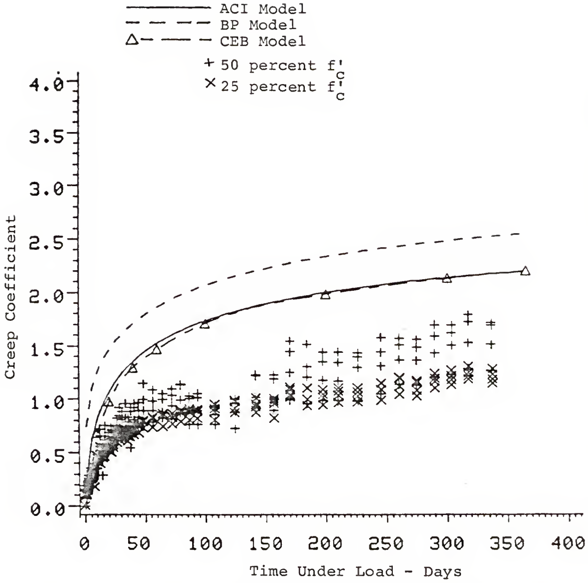


Figure 6.5. Comparison of Creep Prediction Models with Miami Oolite Concrete Stored in the Air Conditioned Room and Loaded to 25 and 50 Percent of f'_c at 28 Days.

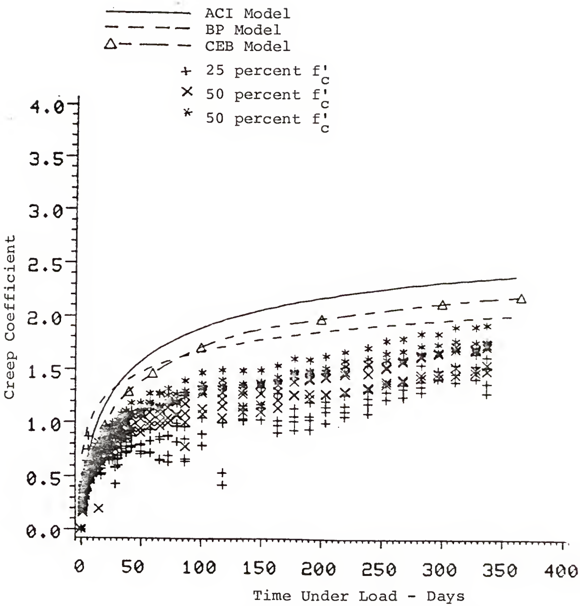


Figure 6.6. Comparison of Creep Prediction Models with Brooksville Concrete Stored in the Air Conditioned Room and Loaded at 28 Days to 25 and 50 (2 data sets) Percent of f'_c.

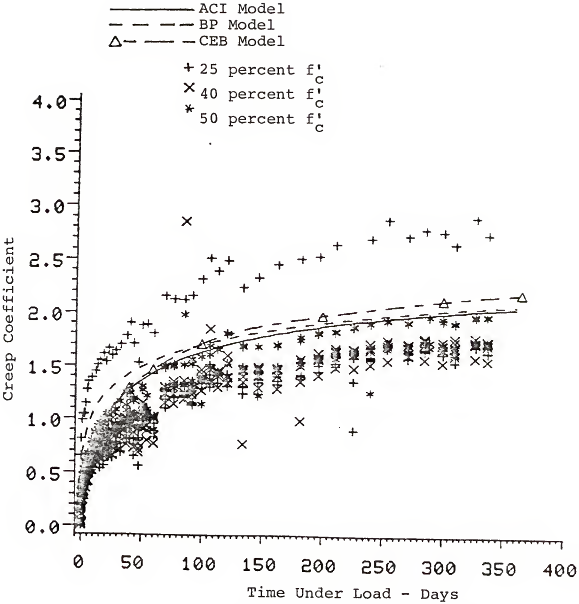


Figure 6.7. Comparison of Creep Prediction Models with Ft. Thompson Concrete Stored in the Air Conditioned Room and Loaded at 28 Days to 25, 40, and 50 Percent of f'_c.

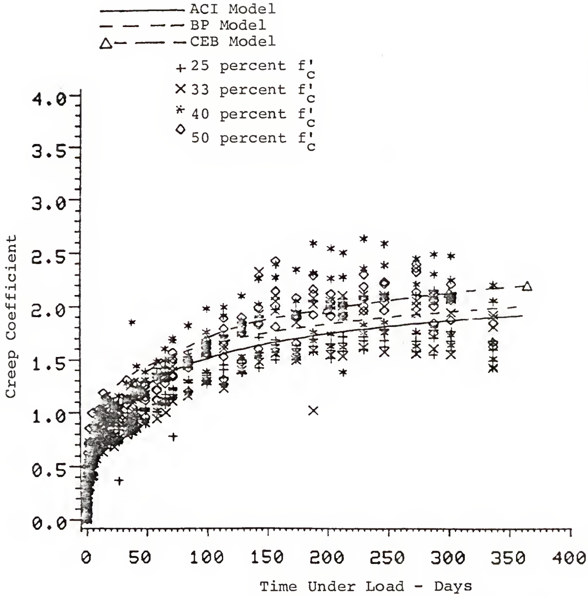


Figure 6.8. Comparison of Creep Prediction Models with Calera Concrete Stored in the Air Conditioned Room and Loaded at 28 Days to 25, 33, 40, and 50 Percent of f'_c.

The comparison of the prediction models with the Calera concrete at four stress levels (25, 33, 40, and 50 percent of f'_c) is shown in Figure 6.8. All the model predictions fit the data and are within the data scatter. The models do not appear to be predicting the shape of the curve very well after about 200 days when the experimental data curve peaks out and begins a decline. It must be remembered that the Calera concrete stored in the air conditioned room is a-typical in that the compressive strength was declining slightly with increasing concrete age (Figure 5.1.(a).). The ACI model predictions are the best or tied in three out of the four concretes (Miami Oolite, Ft. Thompson, and Calera).

Moist room storage

Figures 6.9, 6.10, 6.11, and 6.12 show the comparison of the creep prediction models with the Miami Oolite, Brooksville, Ft. Thompson, and Calera concretes, respectively. The Miami Oolite had only one stress level applied at 50 percent of f'_c for samples stored in the moist room and loaded at 28 days. The CEB model predictions fit the experimental results best and are within the data scatter. The BP model predictions are 41 percent greater than the average of the experimental creep coefficients and the ACI model is 87 percent greater than the average of the experimental creep coefficients.

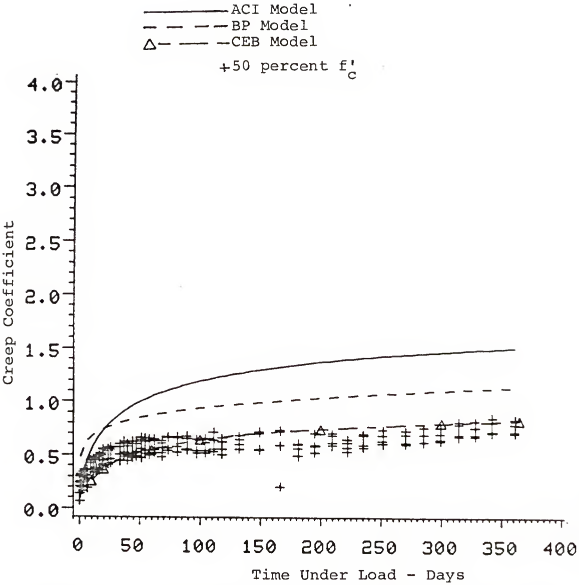


Figure 6.9. Comparison of Creep Prediction Models with Miami Oolite Concrete Stored in the Moist Room and Loaded at 28 Days to 50 Percent of f'_c.

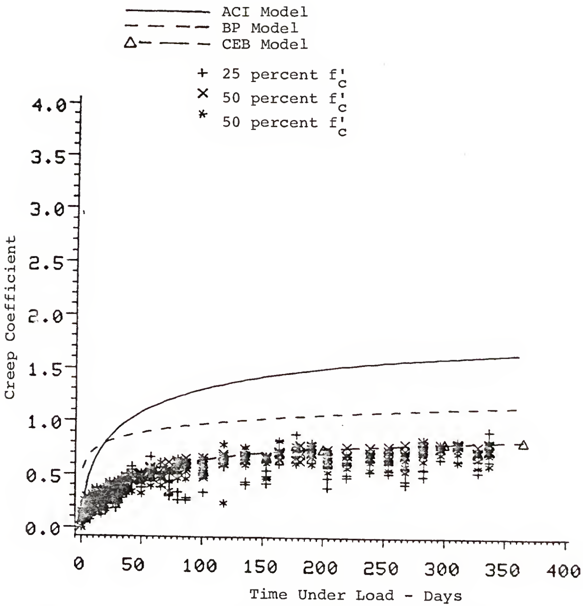


Figure 6.10. Comparison of Creep Prediction Models with Brooksville Concrete Stored in the Moist Room and Loaded at 28 Days to 25 and 50 (2 data sets) Percent of f'_c.

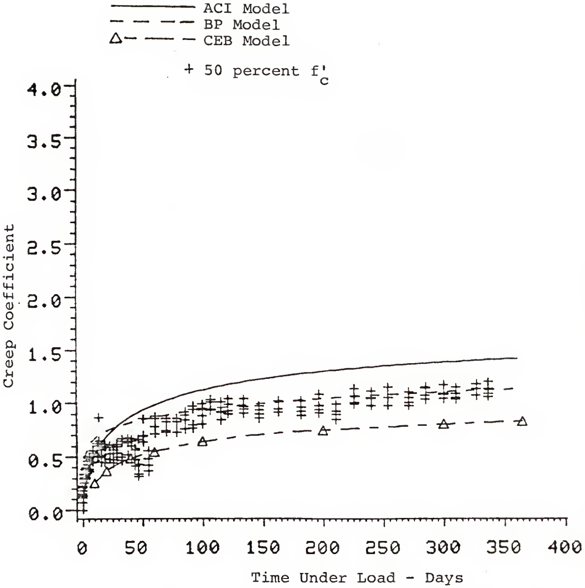


Figure 6.11. Comparison of Creep Prediction Models with Ft. Thompson Concrete Stored in the Moist Room and Loaded at 28 Days to 50 Percent f'_c.

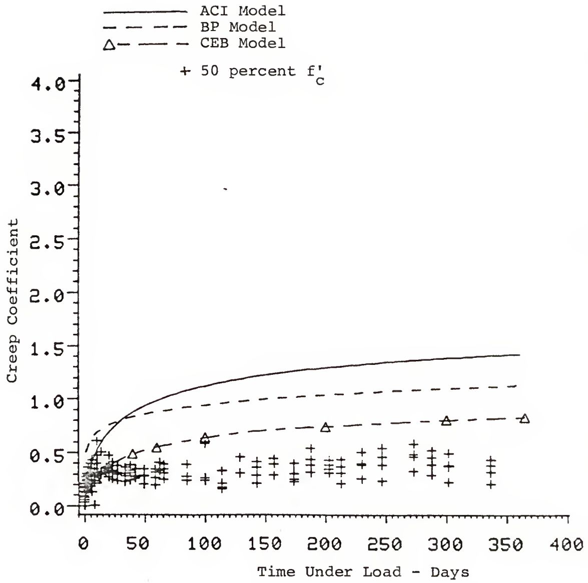


Figure 6.12. Comparison of Creep Prediction Models with Calera Concrete Stored in the Moist Room and Loaded at 28 Days to 50 Percent of f'_c.

The behavior of the creep prediction models when compared with the experimental results of the Brooksville concrete (Figure 6.10) is very similar to the behavior with respect to the Miami Oolite concrete. The CEB model predictions fall within the scatter of the experimental data from the two stress levels (25 and 50 percent f'_c) that are shown in this figure. The BP model predictions are 41 percent greater than the average of the experimental creep coefficients which is the same as with the Miami Oolite concrete. The ACI model predictions are 102 percent greater than the average of the experimental creep coefficients at the end of the testing period.

In Figure 6.11 the BP model shows the best fit of the three prediction models. Figure 6.11 shows the comparison of the three prediction models with Ft. Thompson concrete stored in the moist room and loaded at 28 days to 50 percent of f'_c. The predictions of the ACI and the CEB models are equidistant to the average of the experimental creep coefficients at the end of the test period. The ACI model predictions are 31 percent greater and the CEB model predictions are 31 percent less than the average creep coefficients at the end of the test period.

None of the prediction models are very close to the experimental creep coefficients at the end of the test period in Figure 6.12. Figure 6.12 compares the creep prediction models with Calera concrete stored in the moist room and loaded to 50 percent of f_c' at 28 days. The CEB model fits best and is 78 percent greater than the experimental creep coefficients at the end of the test period. The BP model predictions are 142 percent greater and the ACI model predictions are 200 percent greater than the average creep coefficients of the experimental data.

Age at Loading of 90 Days

Only two of the concretes, Miami Oolite and Brooksville, were tested with loadings at a concrete age of 90 days. The comparison of the creep prediction models with Miami Oolite concrete stored in the air conditioned room and loaded at 90 days to 50 percent of f_c' is shown in Figure 6.13. The CEB model predictions show the best fit but the curve is still well outside the data scatter of the experimental results. The CEB model predictions are 34 percent greater than the average of the experimental creep coefficients at the end of the test period. The BP model and ACI model predictions are 66 and 101 percent greater, respectively, than the average experimental creep coefficients at the end of the test period.

The model predictions show the same relative goodness of fit with the experimental data from the Brooksville concrete as was shown with the experimental data from the Miami Oolite concrete (Figure 6.14). The CEB model predictions fall well within the scatter of the experimental creep coefficients data. The BP model predictions were 16 percent greater than

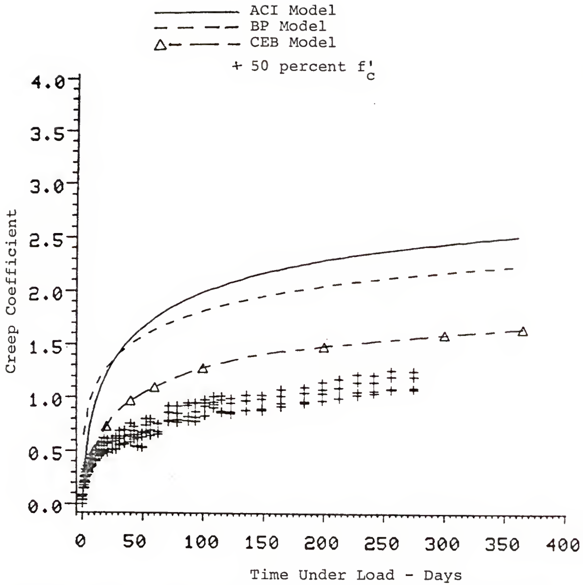


Figure 6.13. Comparison of Creep Prediction Models with Miami Oolite Concrete Stored in the Air Conditioned Room and Loaded at 90 Days to 50 Percent of f'_c.

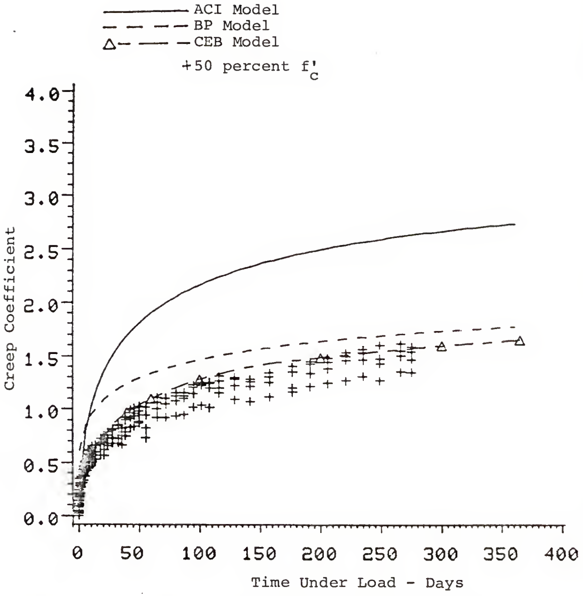


Figure 6.14. Comparison of Creep Prediction Models with Brooksville Concrete Stored in the Air Conditioned Room and Loaded at 90 Days to 50 Percent f'_c.

the average experimental creep coefficients. The ACI model predictions are 75 percent greater than the average of the experimental creep coefficients at the end of the test period.

Improvements to the ACI Creep Model

Table 6.1 lists the amount the ACI creep model predictions were greater than or less than the average of the actual experimental data. Because the ACI model (1) only considers concrete that has been loaded after a concrete age of seven days, the three and seven day loadings have not been considered. The most logical place to adjust the model would be in the value of the ultimate creep coefficient for specimens stored in the air conditioned environment and loaded at 28 days whose mean values are 26 percent greater than the ACI model. This action would require the lowering of the ultimate creep coefficient by 26 percent to a value of 1.74. Table 6.2 lists the amount the ACI model predictions using an ultimate creep coefficient of 1.74 is greater than or less than the average of the actual experimental data at the end of the test period. Though a definite improvements, the predicted creep coefficients still range from 26 percent less than to 122 percent greater than the experimental creep coefficients at the end of the test period. Further improvements would involve altering basic parameters used in the ACI prediction model which is beyond the scope of this study.

Table 6.1. Comparison of ACI Model to Experimental Data Using an Ultimate Creep Coefficient of 2.35; (+) Greater Than, (-) Less Than Experimental Data.

Concrete	Percent Difference at an Age of Loading 3/7 Days (A/C)	 28 Days (A/C)	 28 Days (M/R)	 90 Days (A/C)
Miami Oolite	+104	+47	+87	+101
Brooksville	0	+38	+102	+75
Ft. Thompson	+12	+18	+31	--
Calera	-23	0	+200	--

Table 6.2. Comparison of ACI Model to Experimental Data Using an Ultimate Creep Coefficient of 1.74; (+) Greater Than, (-) Less Than Experimental Data.

Concrete	Percentage Difference at an Age of Loading 28 Days (A/C)	 28 Days (M/R)	 90 Days (A/C)
Miami Oolite	+10	+39	+50
Brooksville	+2	+51	+30
Ft. Thompson	-11	-8	---
Calera	-26	+122	---

Comparison of Shrinkage Models

The three shrinkage models used are the same as described in Chapter II, the ACI Committee 209 model, the CEB-FIP model and the simplified Bazant Panula (BP) model. The same assumptions that applied to creep were also applied to the shrinkage prediction models, namely 60 percent relative humidity. All the shrinkage models gave predictions in strains so no conversion was necessary. Because the shrinkage from the moist room specimens was zero, the prediction models were not applied to the moist room. Thus, this section will only deal with the comparison of shrinkage model and experimental data for specimens stored in the air conditioned room.

Figure 6.15 shows a comparison of the three shrinkage models with the Miami Oolite concrete. Shrinkage testing of the Miami Oolite concrete was begun at a concrete age of 3 days and thus all the models have been adjusted for starting at 3 days. The CEB model predictions are 31 percent lower than the experimental data. The CEB model predictions will not change with a change in the aggregate because there is no such provision in its methodology. The ACI model predictions have the best fit and parallel the larger strains of the experimental data. The BP model predictions are 51 percent greater than the average of the experimental strains at the end of the test period.

The shrinkage curve for the Brooksville concrete, Figure 6.16, follows a similar pattern except that the ACI model parallels the lower experimental shrinkage strain data. The BP model predictions are 59 percent greater than the experimental strains. The CEB model predictions are 42 percent

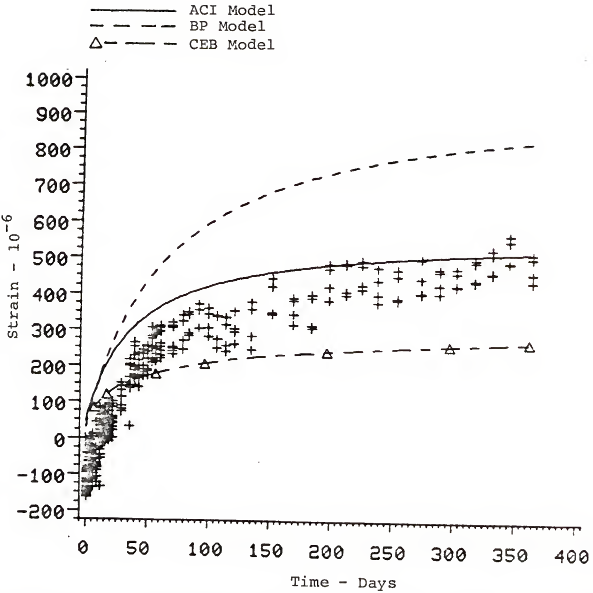


Figure 6.15. Comparison of Shrinkage Model Predictions with Miami Oolite Concrete Stored in the Air Conditioned Room.

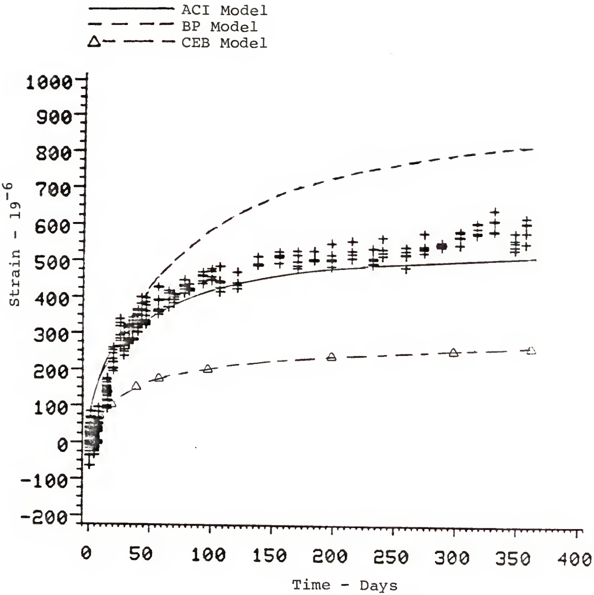


Figure 6.16. Comparison of Shrinkage Model Predictions with Brooksville Concrete Stored in the Air Conditioned Room.

lower than the average experimental strains at the end of the testing period.

The same pattern exists with the Ft. Thompson concrete, Figure 6.17, as did with the Miami Oolite and the Brooksville concretes. The relative position of the three model predictions remains unchanged. The ACI model gives a relatively good prediction paralleling the lower experimental strain readings. The BP model predictions are 47 percent greater and the CEB model predictions are 46 percent lower than the average experimental strains at the end of the test period.

The comparison of the shrinkage predictions of the three models with the experimental strain from the Calera concrete, as shown in Figure 6.18, is similar to that for the other three aggregates. The ACI model predictions are relatively good and lie within the scatter of the experimental data. The BP model predictions are 73 percent greater and the CEB model predictions are 29 percent lower than the average experimental strains at the end of the test period.

Burgers Model

The Burgers model that will be utilized here is the same model that was described in Chapter II. The equation for this model is:

$$\text{Total Strain} = \varepsilon = \frac{\sigma}{E_1} + \frac{\sigma}{E_2}(1-e^{-t/\tau}) + \frac{\sigma}{\eta_3}t$$

$$\text{Where: } \frac{1}{\tau} = \frac{E_2}{\eta_2}$$

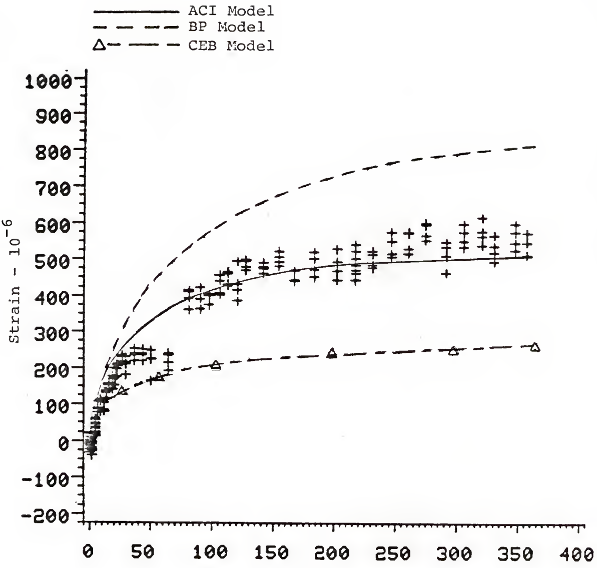


Figure 6.17. Comparison of Shrinkage Model Predictions with Ft. Thompson Concrete Stored in the Air Conditioned Room.

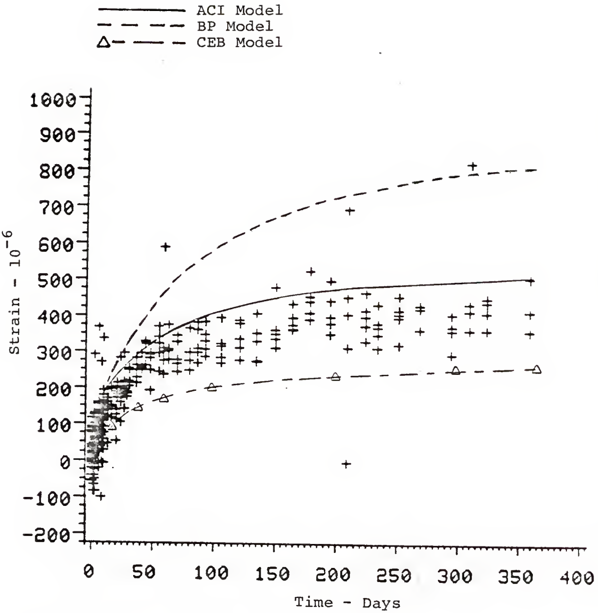


Figure 6.18. Comparison of Shrinkage Model Predictions with Calera Concrete Stored in the Air Conditioned Room.

The first term in the above equation, which is ε_0 in Figure 2.3, was found by taking the mean of the initial strains after loading. The third term, ε_c, in the above equation along with the creep rate, $\frac{\sigma}{3}$, was found by linearly interpolating all strains recorded after 120 days of the load being applied. The interpolation was done using SAS and the results are provided in Table 6.3. The intercept is the sum of ε_0 and ε_d (Figure 2.3). The value of the third term in the equation was found by multiplying the creep rate times the number of days under load. Due to very low R^2 values the creep rates for Calera concrete, except the 7 day specimens, was not reliable and was not utilized. Table 6.4 summarizes the values of ε_0, ε_d, and ε_c.

In order to find the values of ε_2 and η_2, the equation, using the known values for ε_0 and ε_c, was fitted to the data using the nonlinear DUD method of SAS. The results are shown in Table 6.5. The values of ε_1 and η_3 were found from the values of ε_0 and ε_c, respectively, along with the corresponding value of applied stress. Values of ε_1, ε_2, η_2, and η_3 are shown in Table 6.6.

Table 6.3. Creep Rate and Strain Intercept.

Concrete	Storage*	Age of Loading (Days)	Stress Level (% f'_c)	Creep Rate (10^{-6} in/in)/ Day	Std. Error of Estimate of Creep rate	Intercept (10^{-6} in/in)	Std. Error of Estimate of Intercept	R^2
Miami Oolite	A/C	3	50	0.7524	0.1575	1331.86	40.16	0.269
	A/C	28	25	1.1955	0.0727	765.54	17.34	0.823
	A/C	28	50	1.3078	0.2115	1640.73	50.42	0.397
	A/C	90	50	2.1566	0.1825	2188.39	37.05	0.769
	M/R	28	50	0.5925	0.1192	1185.79	28.42	0.299
Brooksville	A/C	7	50	1.7892	0.2327	1703.74	58.82	0.472
	A/C	28	25	0.8661	0.1078	553.20	26.49	0.527
	A/C	28	50	1.8862	0.1822	1730.34	44.76	0.649
	A/C	28	50	2.0770	0.0808	2062.46	19.85	0.919
	A/C	90	50	2.0508	0.3290	1865.12	69.09	0.481
	M/R	28	25	0.2112	0.0992	464.01	24.39	0.772
	M/R	28	50	0.4260	0.2111	1368.40	51.93	0.066
	M/R	28	50	0.1845	0.1439	1143.63	35.36	0.028
Ft. Thompson	A/C	7	50	1.2810	0.1382	2185.01	33.80	0.537
	A/C	28	25	0.8090	0.1853	933.22	44.74	0.235
	A/C	28	40	1.3978	0.2718	1613.24	65.61	0.299
	A/C	28	50	1.4798	0.2494	2383.79	60.26	0.362
	M/R	28	50	0.7433	0.1144	1521.87	27.61	0.405
Calera	A/C	7	50	1.0217	0.1950	1786.63	46.43	0.321
	A/C	28	25	0.1828	0.2366	698.84	54.52	0.012
	A/C	28	33	0.2362	0.1731	1054.21	39.90	0.036
	A/C	28	40	0.1285	0.2158	1313.75	49.74	0.007
	A/C	28	50	0.5269	0.2659	1858.54	61.31	0.073
	M/R	28	50	0.3760	0.1395	977.11	32.15	0.127

*M/R = Moist Room Storage   A/C = Air Conditioned Room Storage

Table 6.4. Burgers Model Strains.

Concrete	Storage	Age of Loading (Days)	Stress Level (% f'_c)	ε_0	$\varepsilon_d + \varepsilon_0$	ε_d	ε_c
Miami Oolite	A/C	3	50	791.22	1331.86	540.64	182.08
	A/C	28	25	454.75	765.86	310.99	259.42
	A/C	28	50	936.61	1640.73	704.14	223.68
	A/C	90	50	1158.22	2188.39	1030.17	334.27
	M/R	28	50	790.25	1185.79	395.54	128.57
Brooksville	A/C	7	50	737.25	1703.74	966.49	425.83
	A/C	28	25	338.03	553.20	215.17	187.93
	A/C	28	50	902.68	1730.34	827.48	409.31
	A/C	28	50	984.53	2062.46	1077.93	450.71
	A/C	90	50	968.01	1865.12	897.11	317.87
Ft. Thompson	A/C	7	50	1014.82	2185.01	1170.90	304.88
	A/C	28	25	410.07	933.22	523.15	175.55
	A/C	28	40	754.54	1613.24	858.70	303.32
	A/C	28	50	1005.58	2383.79	1378.21	321.12
	M/R	28	50	838.34	1521.87	683.53	161.30
Calera	A/C	7	50	671.21	1786.63	1115.42	243.16

Table 6.5. Burgers Model 0 to 120 Days.

Concrete	Storage	Age of Loading (Days)	Stress Level (% f'_c)	E_2	Asymptotic Standard Error E_2	η_2	Asymptotic Standard Error η_2	R^2
Miami Oolite	A/C	3	50	3.0596	0.0557	6.9442	0.7850	0.967
	A/C	28	25	4.0158	0.0682	50.0508	2.4897	0.987
	A/C	28	50	3.9782	0.0690	52.7136	2.6134	0.985
	A/C	90	50	3.2457	0.0509	18.0954	1.3063	0.979
	M/R	28	50	7.6448	0.2081	127.0475	8.1405	0.966
Brooksville	A/C	7	50	1.8087	0.0336	10.6771	0.8218	0.972
	A/C	28	25	4.7531	0.1171	13.2084	0.9188	0.958
	A/C	28	50	2.4443	0.0472	22.8527	1.3942	0.979
	A/C	28	50	1.9363	0.0245	18.8369	0.7392	0.991
	A/C	90	50	2.5694	0.0463	19.6933	0.4171	0.973
Ft. Thompson	A/C	7	50	2.3208	0.0357	41.0836	1.7714	0.983
	A/C	28	25	3.0158	0.0869	26.4797	3.1495	0.942
	A/C	28	40	2.6681	0.1006	43.9014	4.6902	0.930
	A/C	28	50	2.3350	0.0497	21.3856	1.8392	0.966
	M/R	28		13.9775	0.7344	30.9641	10.9311	0.824
Calera	A/C	7	50	2.6025	0.0503	37.8105	1.9754	0.977

Table 6.6. Burgers Model Coefficients.

Concrete	Storage	Age of Loading	Stress Level	E_1	E_2	η_1	η_3
Miami Oolite	A/C	3	50	2.1903	3.0596	2303.32	6.9442
	A/C	28	25	2.9352	4.0158	1116.50	50.0508
	A/C	28	50	2.8502	3.9782	2041.26	52.7136
	A/C	90	50	1.7624	3.2457	1356.67	18.0954
	M/R	28	50	3.2600	7.6648	4348.00	127.0475
Brooksville	A/C	7	50	2.1708	1.8087	894.47	10.6771
	A/C	28	25	2.8700	4.7531	1120.12	13.2084
	A/C	28	50	2.1490	2.4443	1028.67	22.8527
	A/C	28	50	1.9708	1.9363	934.17	18.8369
	A/C	90	50	2.1129	2.5694	997.33	19.6933
Ft. Thompson	A/C	7	50	0.9828	2.3208	1790.20	41.0836
	A/C	28	25	3.2257	3.0158	1635.04	26.4797
	A/C	28	40	2.8054	2.6681	1514.35	43.9014
	A/C	28	50	2.6308	2.3350	1787.75	21.3856
	M/R	28	50	3.1556	13.9975	3559.14	30.9641
Calera	A/C	7	50	3.3728	2.6025	2215.81	37.8105

Chapter VII
CONCLUSIONS

The long term deformation of concrete is composed of three parts; elastic deformation, shrinkage deformation and creep deformation. This study examined each of these portions of long term concrete deformations.

Elastic Deformations

The elastic deformation was determined by both the initial elastic deformation of the specimens when loaded, and by determining the modulus of elasticity of concrete cylinders from the same mixture. The actual elastic deformations were generally higher (overall by 23 percent) than the deformations predicted by the modulus of elasticity. This effect was observed by ACI (1) and quantified by Bazant (12), who determined that actual elastic deformations were 50 percent greater than those calculated from the static modulus of elasticity.

Shrinkage

The shrinkage characteristics of all the concretes made with the different aggregates were similar. The ACI Committee 209 model produced acceptable shrinkage prediction whereas the B.P. and the C.E.B. models both predicted excessive shrinkages.

Creep

In general the creep of all concrete mixture studied was less than expected. ACI Committee 209 (7) recommended an ultimate creep coefficient (Cu) of 2.35 when there was no creep data available. For all concrete mixtures an ultimate creep coefficient of 1.74, reduced by 26 percent, yielded predictions closer to the actual creep deformations.

Influence of Level of Applied Stress

The effect of the stress level was not significant in any of the specimens studied. This confirms the assumption used in the comparisons using specific creep that the creep is directly proportional to the stress level, e.g. doubling the stress level will double the creep up to about 50 percent of the compressive strength of the concrete. This effect is in concurrence with the results of other studies (39).

Influence of Age of Concrete at Loading

The effect of the age of loading was only important during the early life of the concrete. Except for the concrete made with Ft. Thompson aggregate, specimens from the air conditioned room loaded at 3 and 7 days exhibited greater creep than specimens loaded at 28 or more days. Concrete made with Ft. Thompson aggregate produced essentially the same amount of total creep after about 60 days when compared to specimens loaded at 7 and 28 days.

There was very little difference in the creep of specimens loaded at 28 and 90 days. The effect of increased creep at early ages of loading is attributed by Neville (39) to the hydration of cement which is substantial at an early age.

Influence of Relative Humidity of Storage

The most significant factor influencing creep evaluated in this investigation was the relative humidity of the storage environment. The creep of specimens from the air conditioned room was two and one-half to five times the creep of specimens that had been stored in the moist room.

Influence of Coarse Aggregate

The influence of the coarse aggregate on the creep of the concrete does not indicate that concrete made with the porous Florida aggregates are more susceptible to creep than concrete made with less porous, more dense aggregates. Concrete made with the Miami Oolite and Ft. Thompson aggregates had creep characteristics similar to the concrete made with the hard nonporous Calera aggregate. The concrete made with the Brooksville aggregate did show a greater creep suspectibility but this is probably more attributable to the lower compressive strength of this particular batch of concrete.

Prediction Models

The three prediction models (A.C.I., C.E.B., and B.P.) that were compared against experimental results did not inspire confidence in their use. All of the methods produced good predictions in some of the cases but none of them produced consistent results. The ACI model was improved by modifying the ultimate creep coefficient but the results still gave a large divergence. The use of these methods in actual field conditions should be used with great caution. One thing in favor of the methods is that they usually produce predictions of creep values in excess of those measured in this investigation. The use of these creep prediction models generally result in a conservative estimate of creep.

The attempt was made to fit the Burgers model to the experimental results met with limited results. Due to the Calera concrete not having a linear creep rate during the period of this study, the Burgers model could not be fitted to it. The Burgers model was fitted to the other concrete mixtures but produced a wide scatter of equation coefficients. The fit of the Burgers model could probably be improved if data for a greater time under load was available to better define the linear creep rate.

APPENDIX A
INDIRECT TENSILE CREEP PLOTS

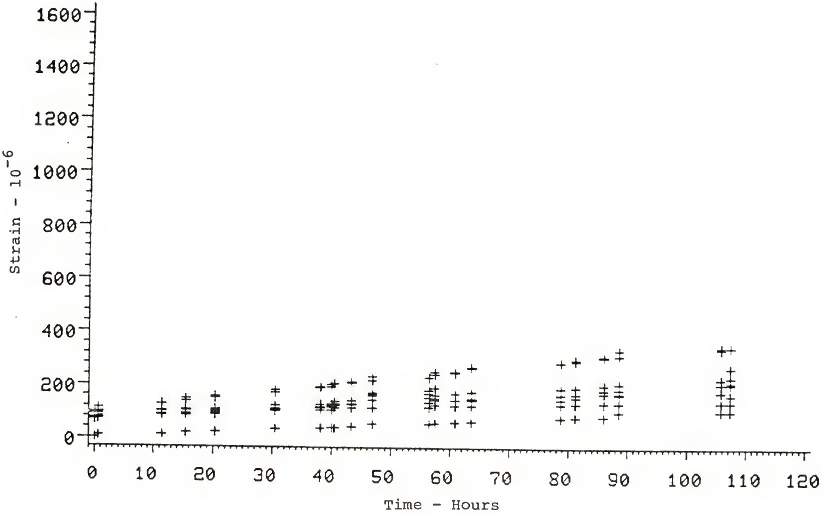


Figure A.1. Indirect Tensile Creep Response for Miami Oolite Concrete Loaded at 3 Days with a Stress of 50% of the Splitting Tensile Strength.

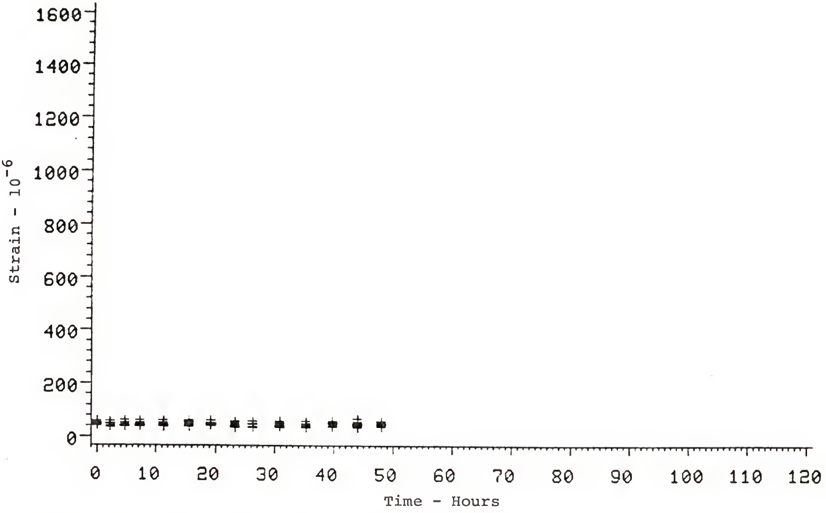


Figure A.2. Indirect Tensile Creep Response for Miami Oolite Concrete Loaded at 28 Days with a Stress of 25% of the Splitting Tensile Strength.

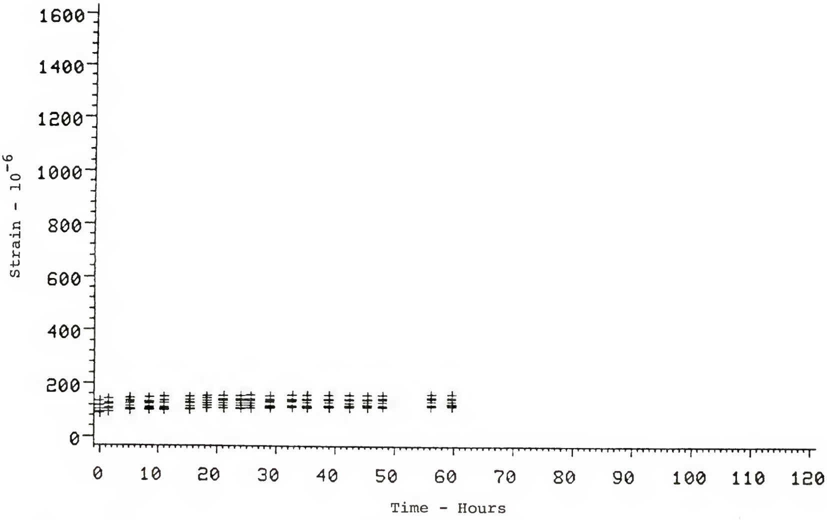


Figure A.3. Indirect Tensile Creep Response for Miami Oolite Concrete Loaded at 28 Days with a Stress of 50% of the Splitting Tensile Strength.

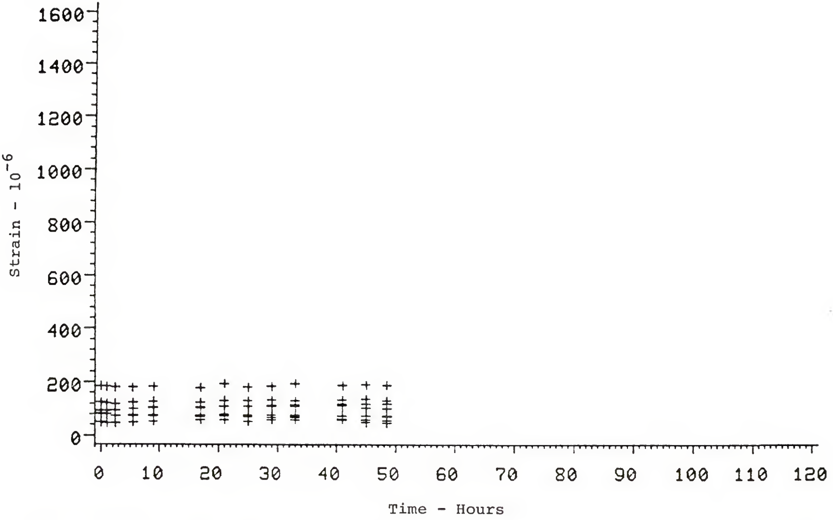


Figure A.4. Indirect tensile Creep Response for Miami Oolite Concrete Loaded at 90 Days with a Stress of 50% of the Splitting Tensile Strength.

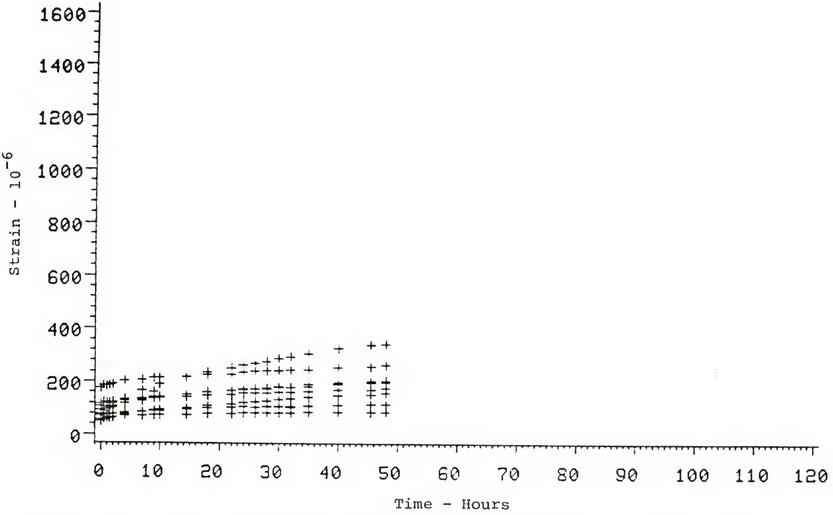


Figure A.5. Indirect Tensile Creep Response for Brooksville Concrete Loaded at 7 Days with a Stress of 50% of the Splitting Tensile Strength.

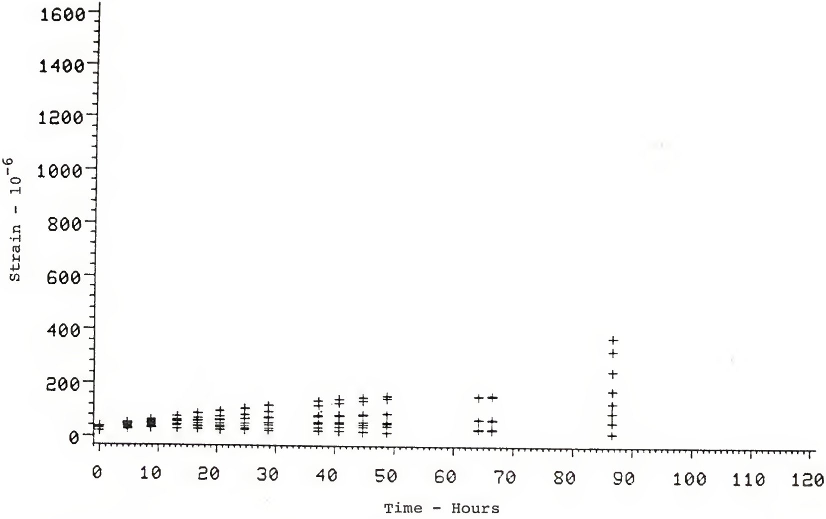


Figure A.6. Indirect Tensile Creep Response for Brooksville Concrete Loaded at 28 Days with a Stress of 25% of the Splitting Tensile Strength.

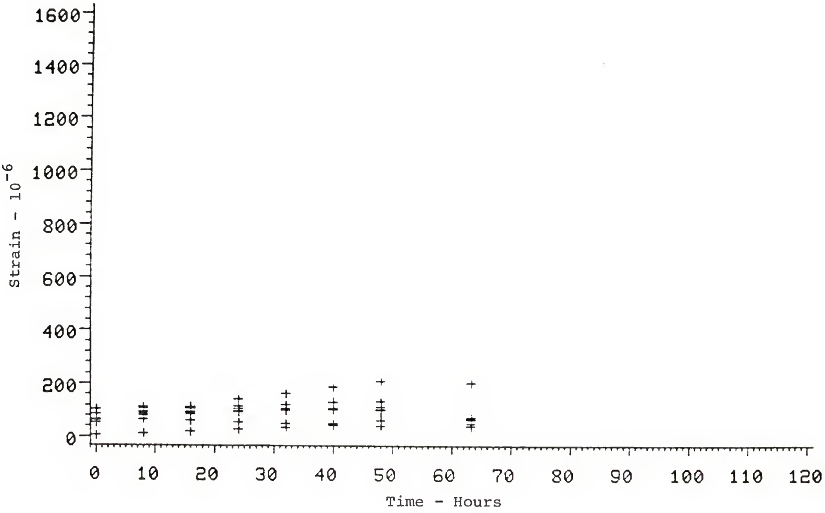


Figure A.7. Indirect Tensile Creep Response for Brooksville Concrete Loaded at 28 Days with a Stress of 50% of the Splitting Tensile Strength.

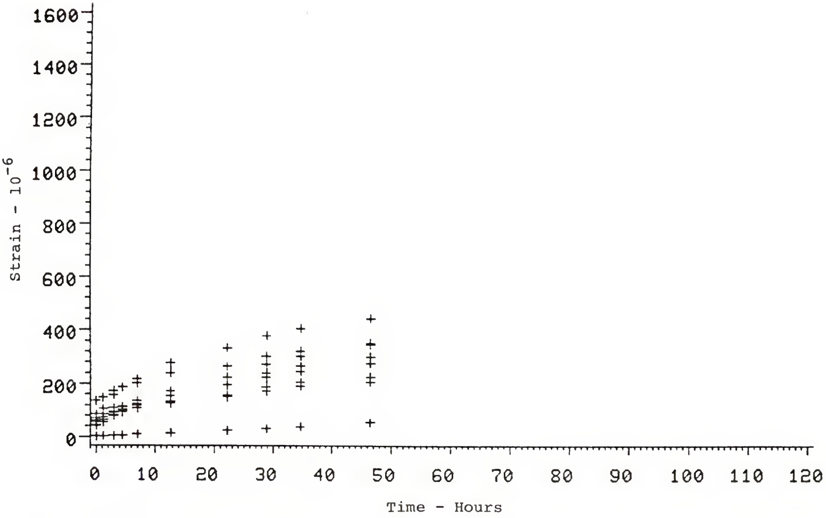


Figure A.8. Indirect Tensile Creep Response for Brooksville Concrete Loaded at 90 Days with a Stress of 50% of the Splitting Tensile Strength.

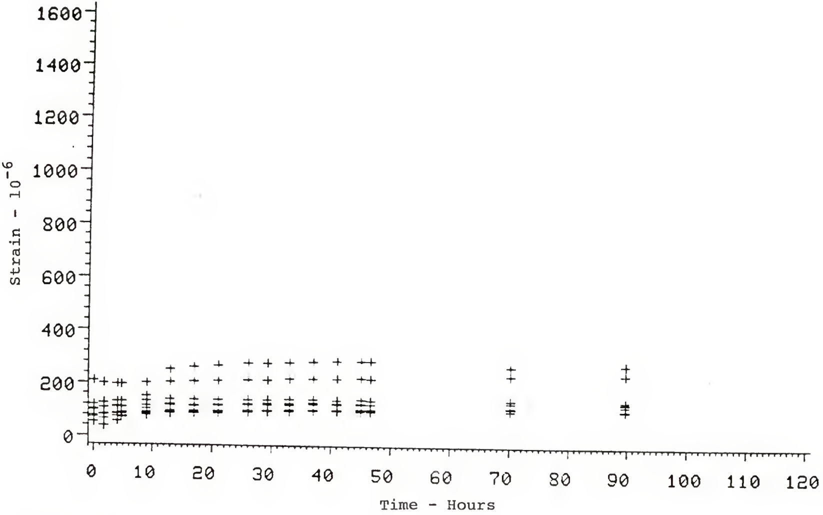


Figure A.9. Indirect Tensile Creep Response for Ft. Thompson Concrete Loaded at 7 Days with a Stress of 50% of the Splitting Tensile Strength.

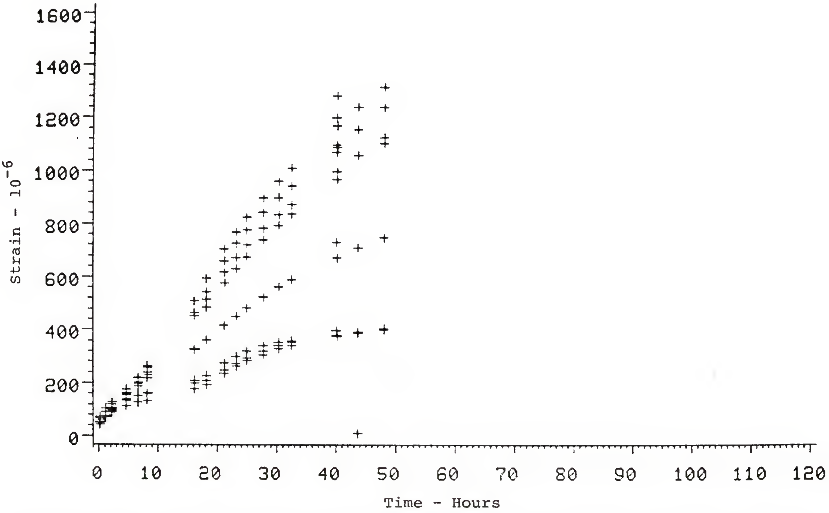


Figure A.10. Indirect Tensile Creep Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 25% of the Splitting Tensile Strength.

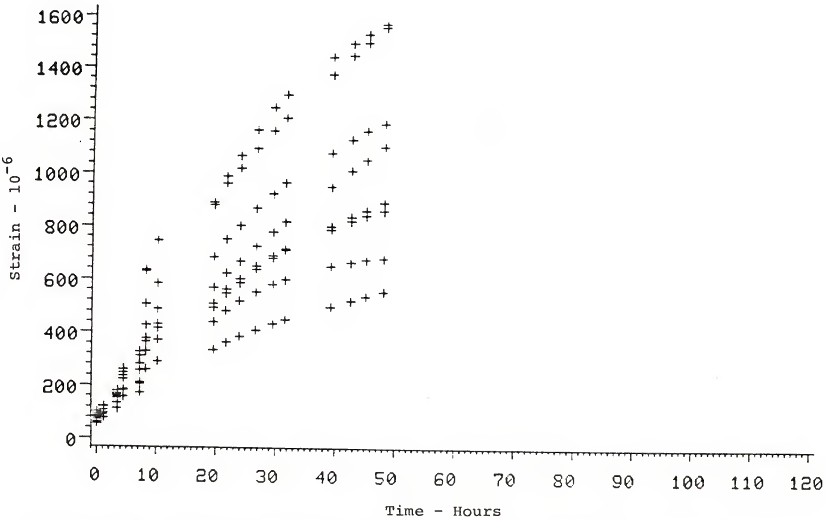


Figure A.11. Indirect Tensile Creep Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 50% of the Splitting Tensile Strength.

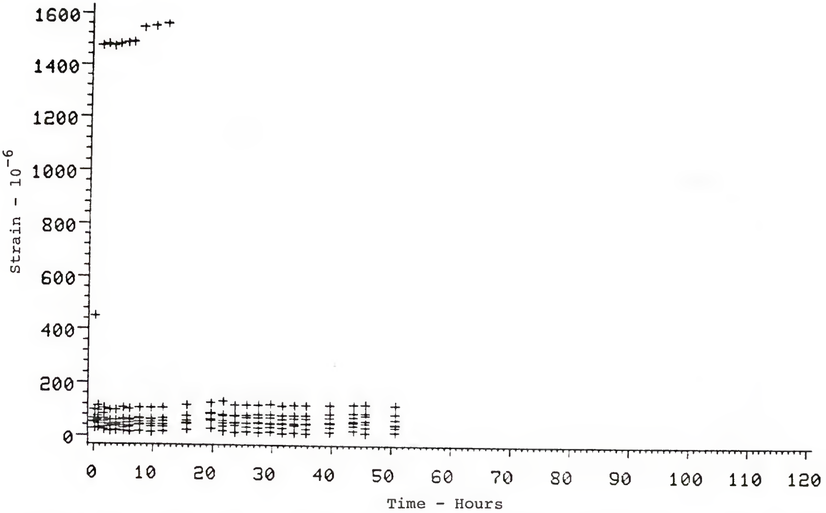


Figure A.12. Indirect Tensile Creep Response for Calera Concrete Loaded at 28 Days with a Stress of 50% of the Splitting Tensile Strength.

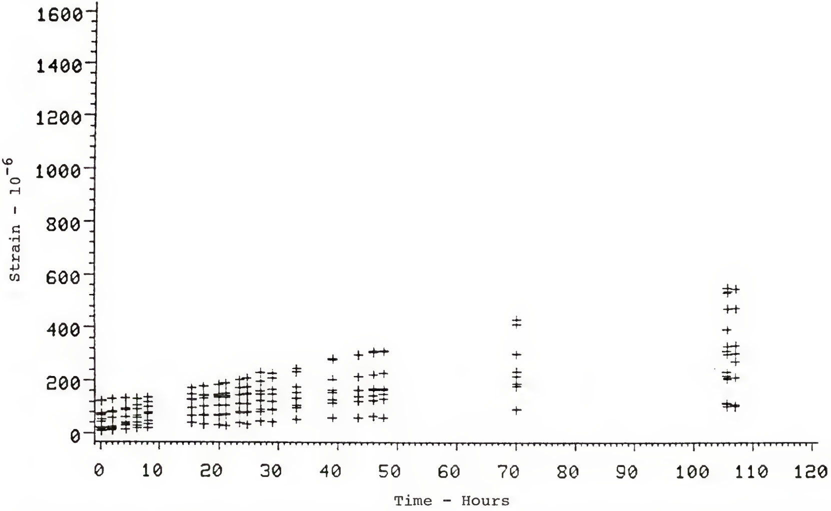


Figure A.13. Indirect Tensile Creep Response for Calera Concrete Loaded at 28 Days with a Stress of 25% of the Splitting Tensile Strength.

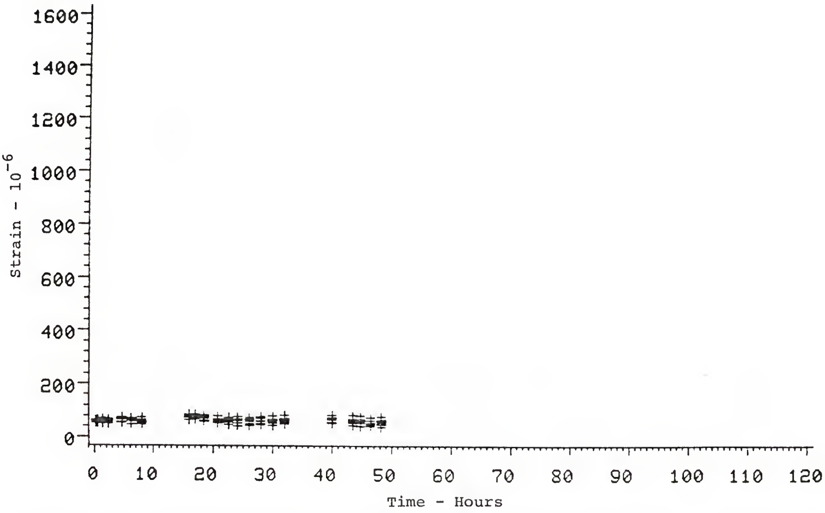


Figure A.14. Indirect Tensile Creep Response for Calera Concrete Loaded at 28 Days with a Stress of 50% of the Splitting Tensile Strength.

APPENDIX B
TOTAL STRAIN PLOTS

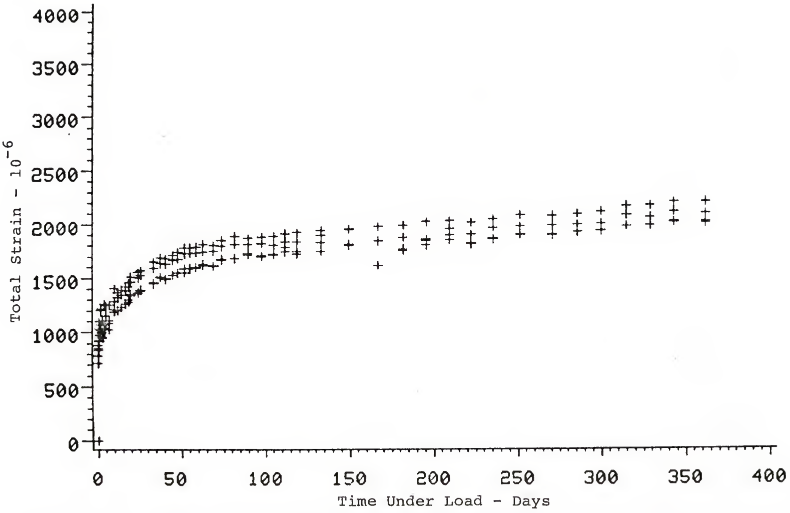


Figure B.1. Total Strain Response for Miami Oolite Concrete Loaded at 3 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

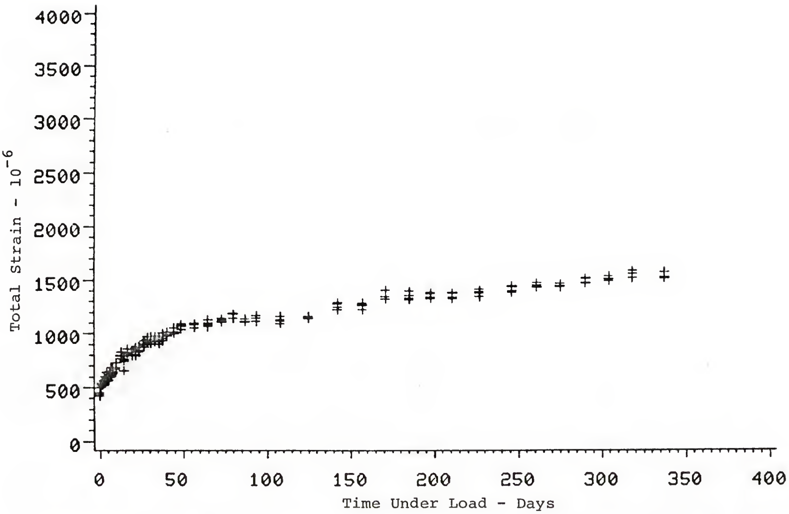


Figure B.2. Total Strain Response for Miami Oolite Concrete Loaded at 28 Days with a Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

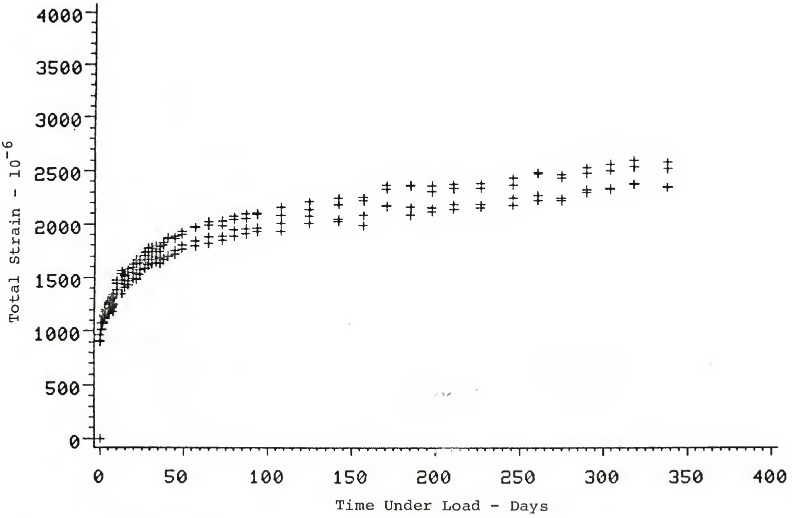


Figure B.3. Total Strain Response for Miami Oolite Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

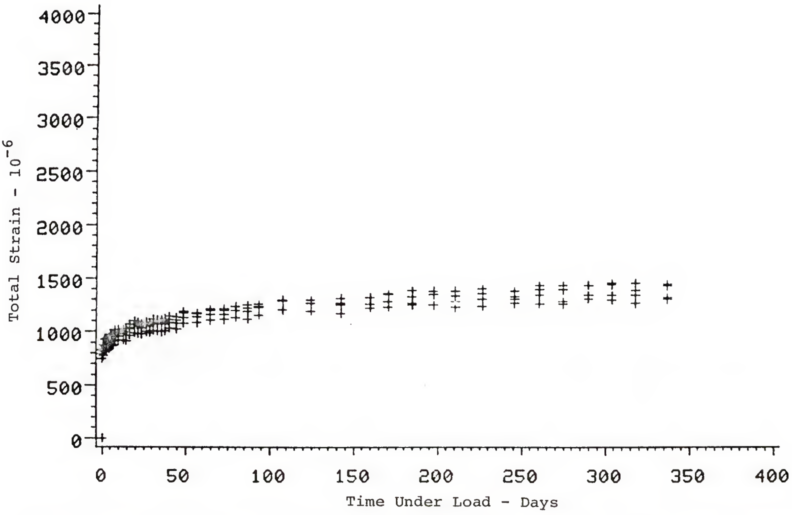


Figure B.4. Total Strain Response for Miami Oolite Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Moist Room.

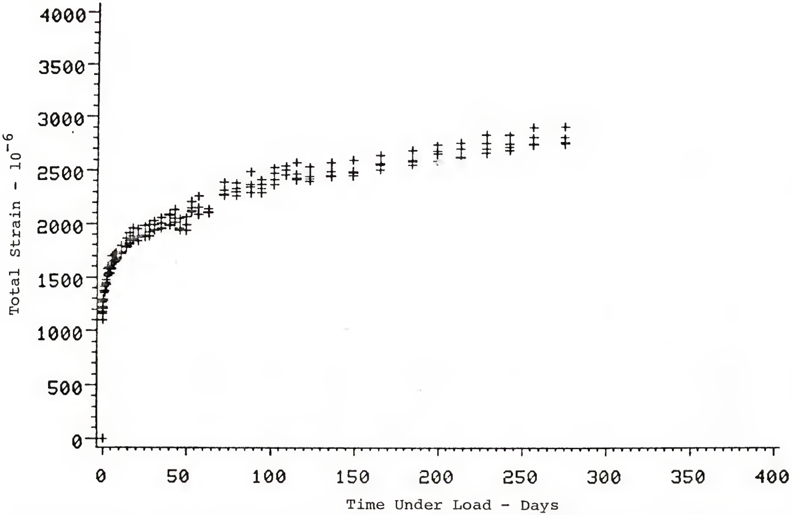


Figure B.5. Total Strain Response for Miami Oolite Concrete Loaded at 90 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

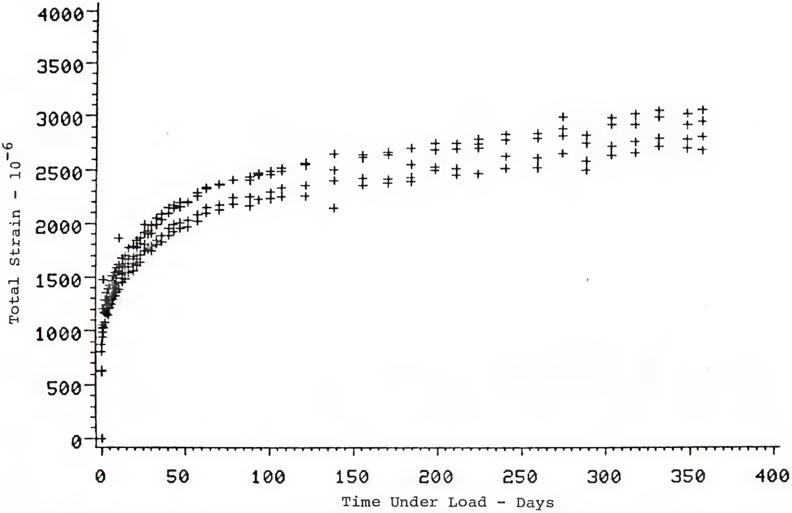


Figure B.6. Total Strain Response for Brooksville Concrete Loaded at 7 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

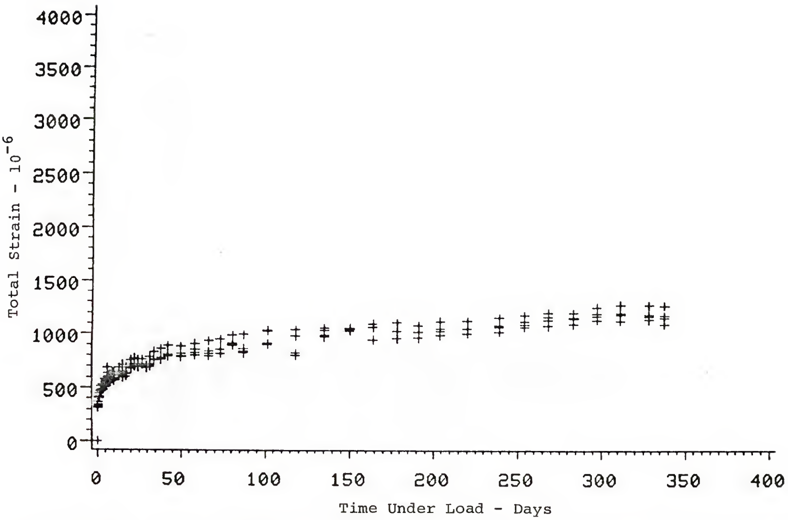


Figure B.7. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

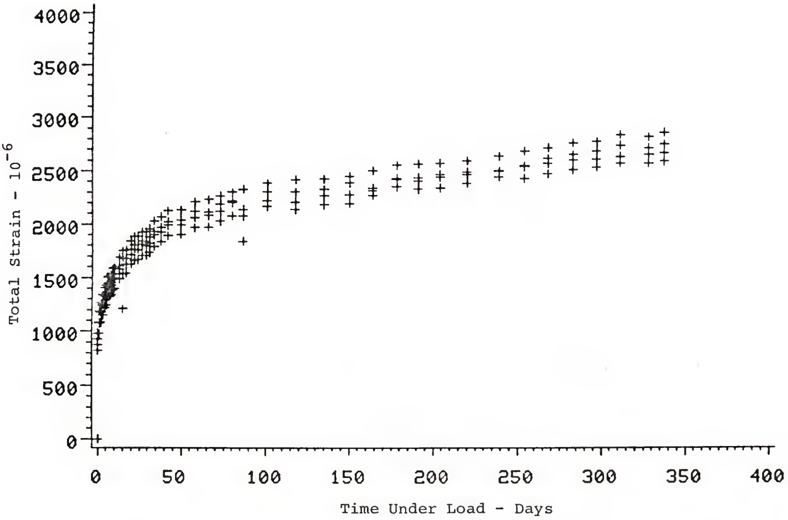


Figure B.8. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

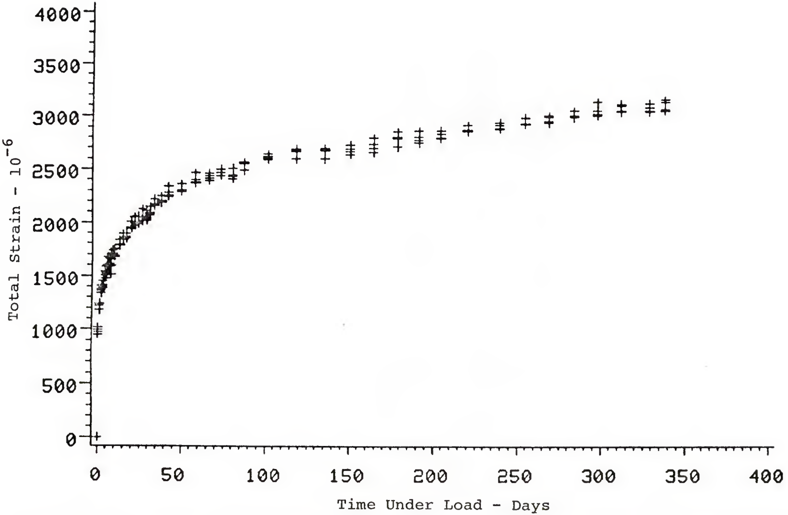


Figure B.9. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

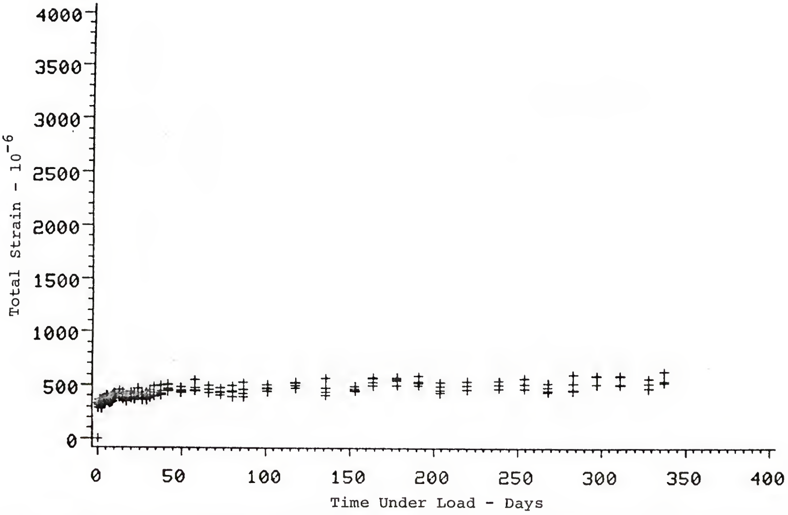


Figure B.10. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 25 Percent f'_c and Stored in the Moist Room.

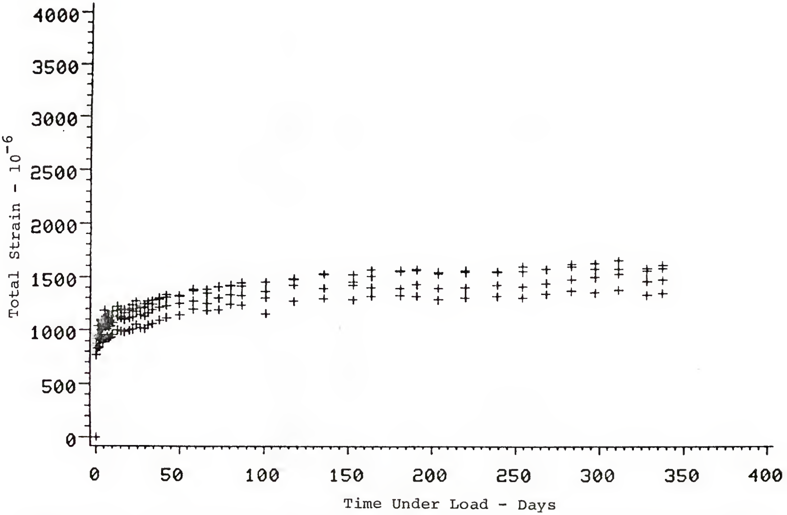


Figure B.11. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Moist Room.

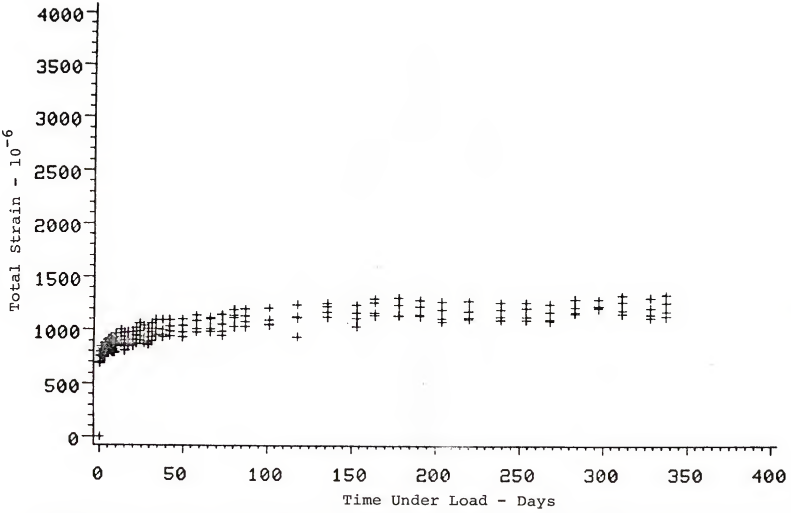


Figure B.12. Total Strain Response for Brooksville Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Moist Room.

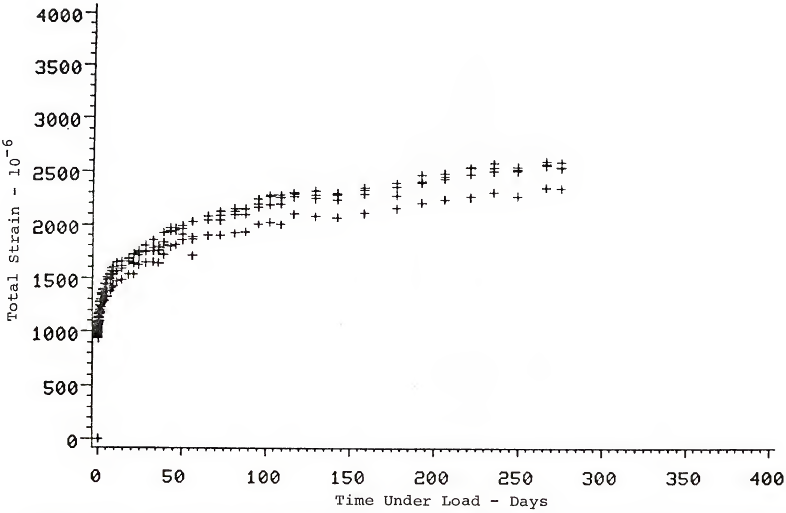


Figure B.13. Total Strain Response for Brooksville Concrete Loaded at 90 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

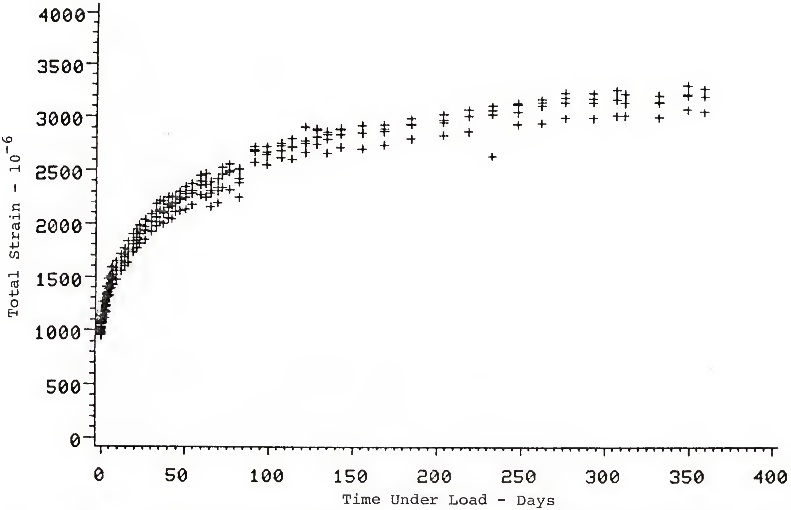


Figure B.14. Total Strain Response for Ft. Thompson Concrete Loaded at 7 Days with a Stress of 50 Percent f_c' and Stored in the Air Conditioned Room.

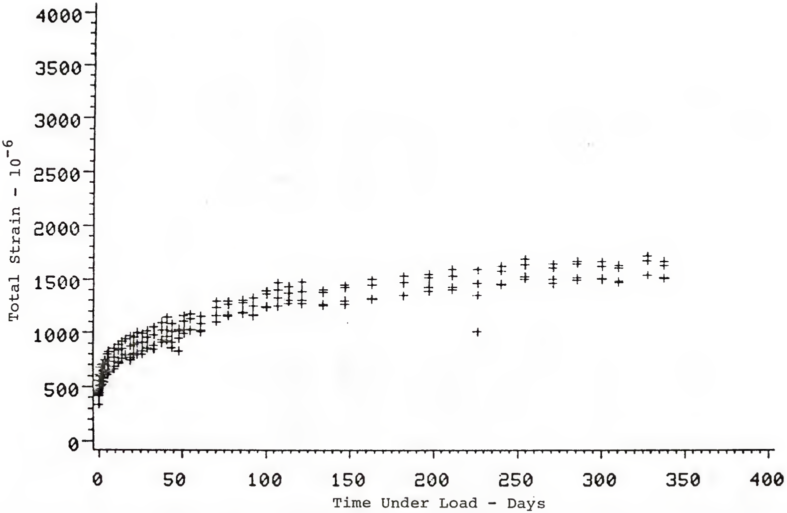


Figure B.15. Total Strain Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 25 Percent f_c' and Stored in the Air Conditioned Room.

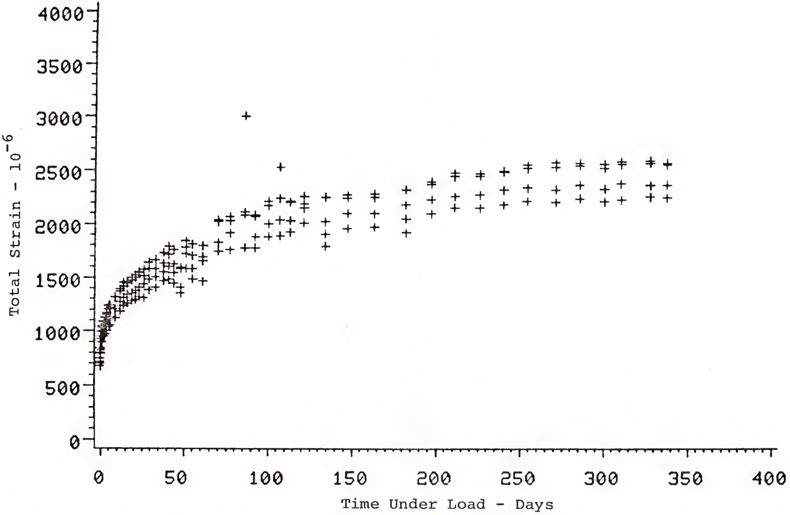


Figure B.16. Total Strain Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 40 Percent f_c' and Stored in the Air Conditioned Room.

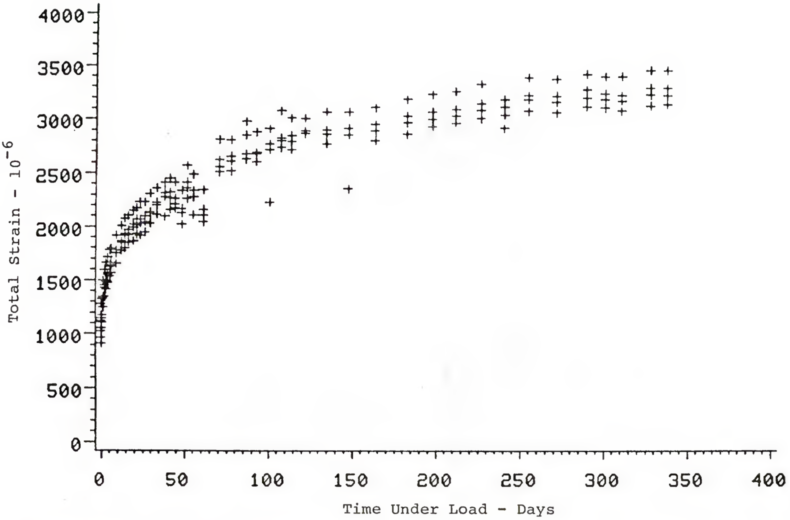


Figure B.17. Total Strain Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

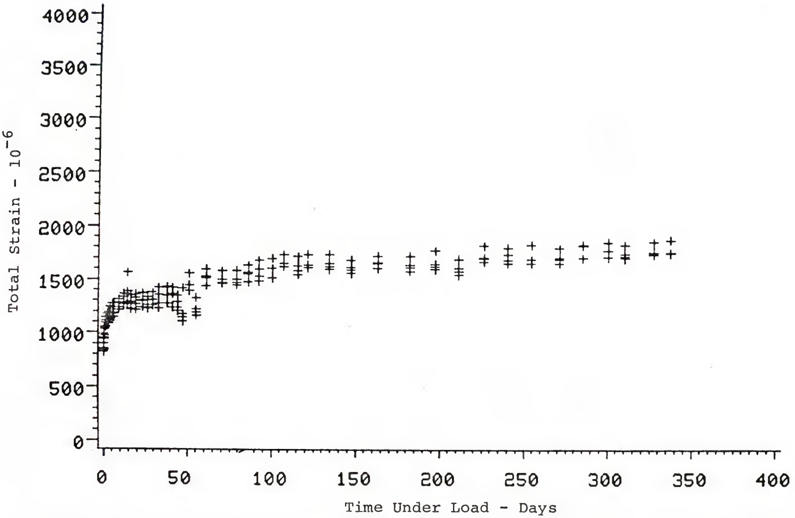


Figure B.18. Total Strain Response for Ft. Thompson Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Moist Room.

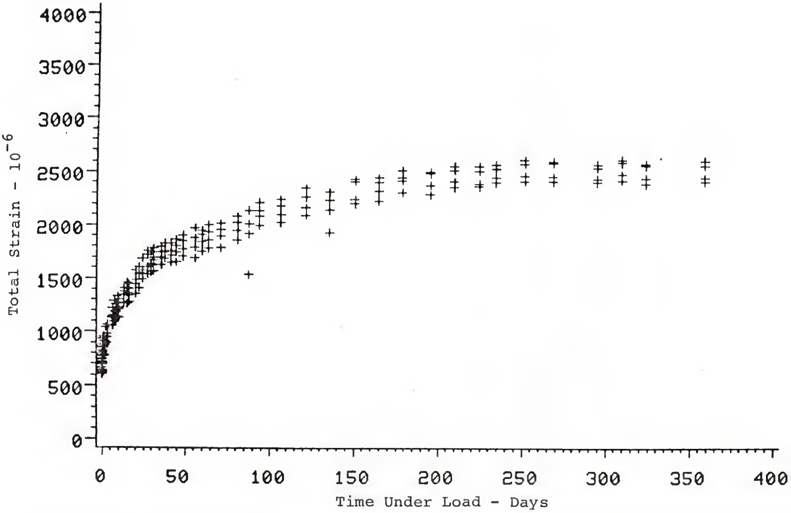


Figure B.19. Total Strain Response for Calera Concrete Loaded at 7 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

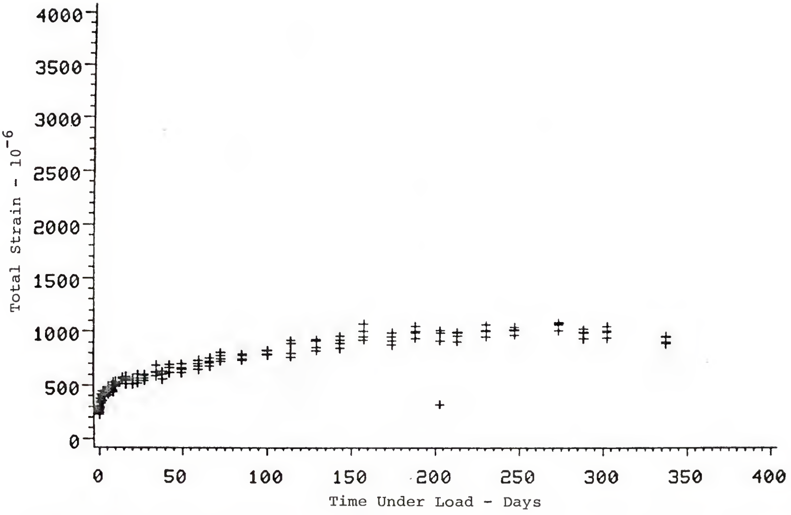


Figure B.20. Total Strain Response for Calera Concrete Loaded at 28 Days with a Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

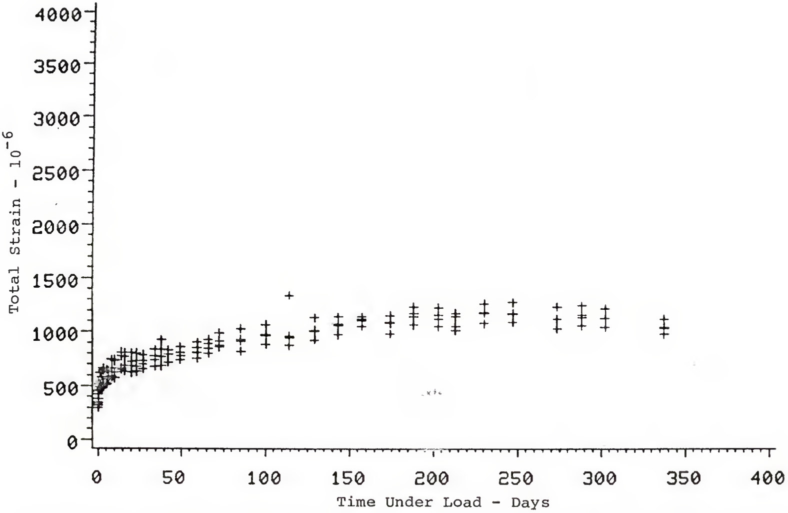


Figure B.21. Total Strain Response for Calera Concrete Loaded at 28 Days with a Stress of 33 Percent f'_c and Stored in the Air Conditioned Room.

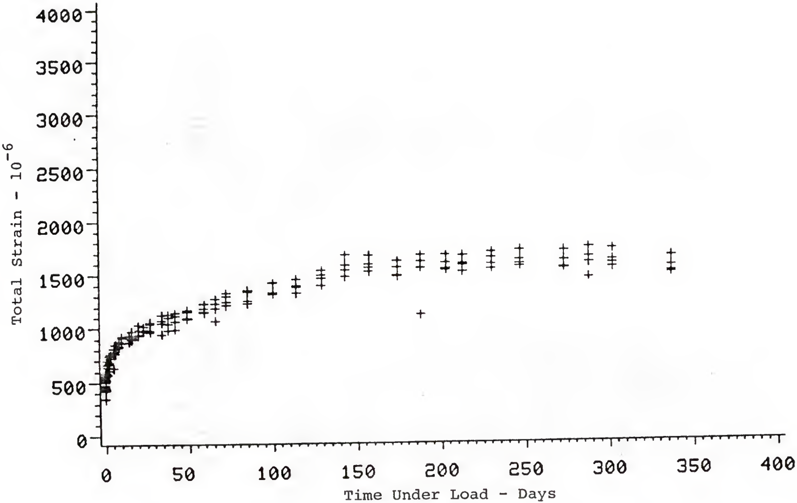


Figure B.22. Total Strain Response for Calera Concrete Loaded at 28 Days with a Stress of 40 Percent f_c' and Stored in the Air Conditioned Room.

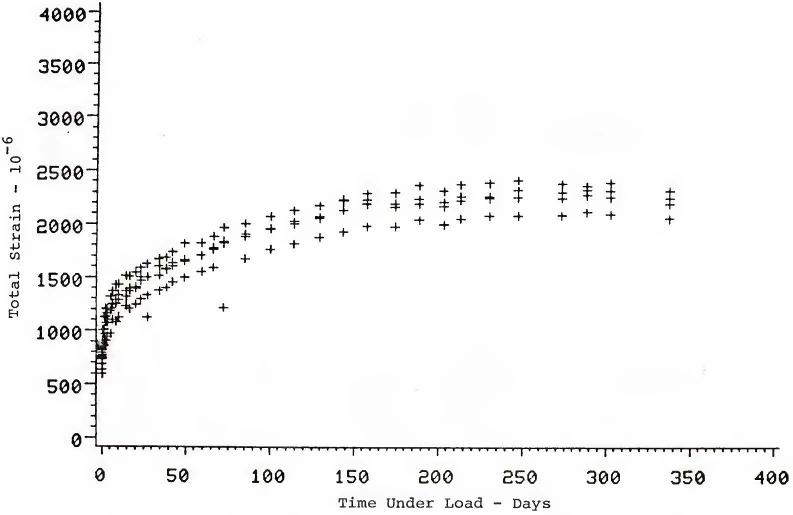


Figure B.23. Total Strain Response for Calera Concrete Loaded at 28 Days with a Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

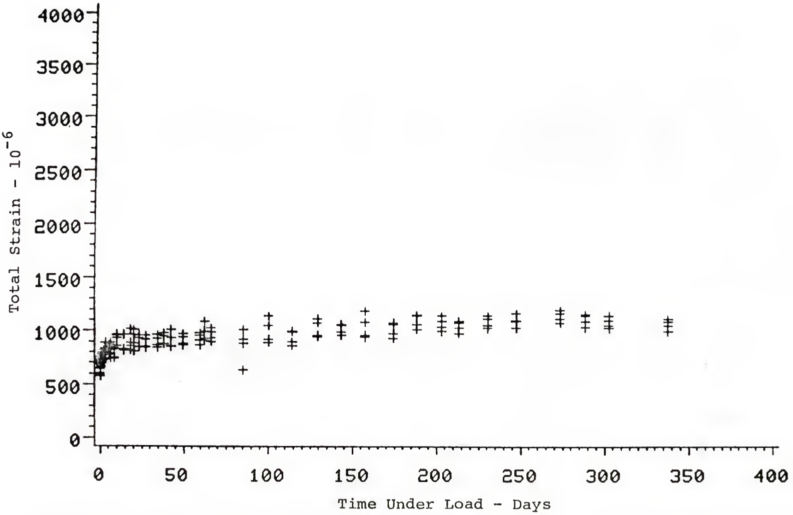


Figure B.24. Total Strain Response for Calera Concrete Loaded at 28 Days with a Stress of 50 Percent f_c' and Stored in the Moist Room.

APPENDIX C
ELASTIC PLUS CREEP PLOTS

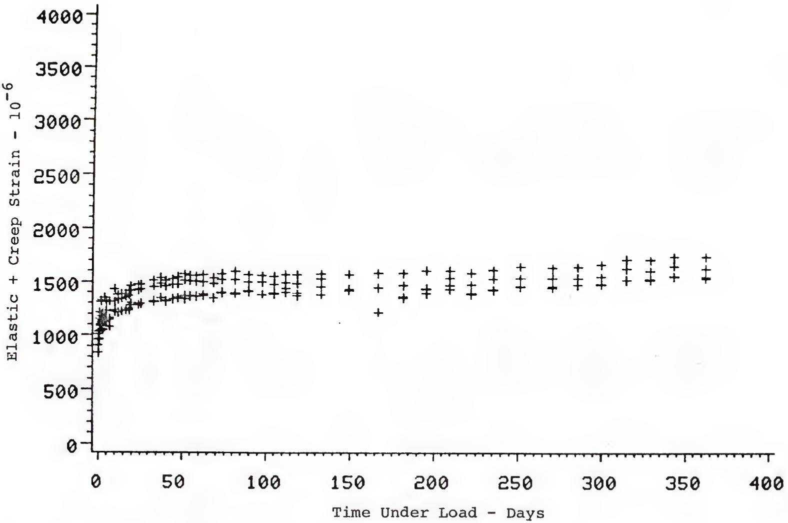


Figure C.1. Elastic Plus Creep Strain Response for Miami Oolite Loaded at 3 Days to Stress of 50 Percent f'_c and stored in the Air Conditioned Room.

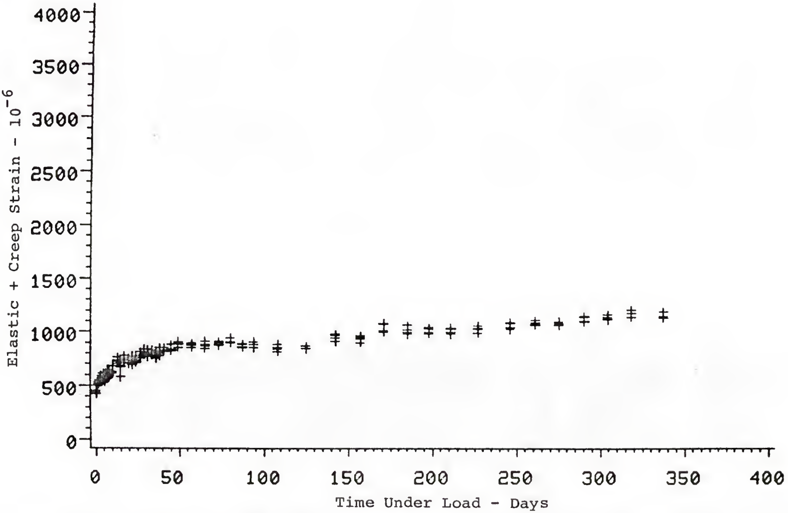


Figure C.2. Elastic Plus Creep Strain Response for Miami Oolite Loaded at 28 Days to Stress of 25 Percent f_c' and Stored in the Air Conditioned Room.

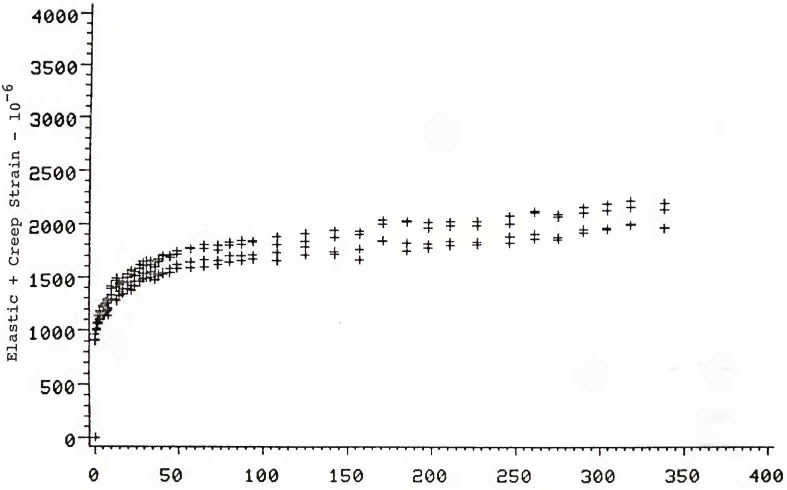


Figure C.3. Elastic Plus Creep Strain Response for Miami Oolite Loaded at 28 Days to Stress of 50 Percent f_c' and Stored in the Air Conditioned Room.

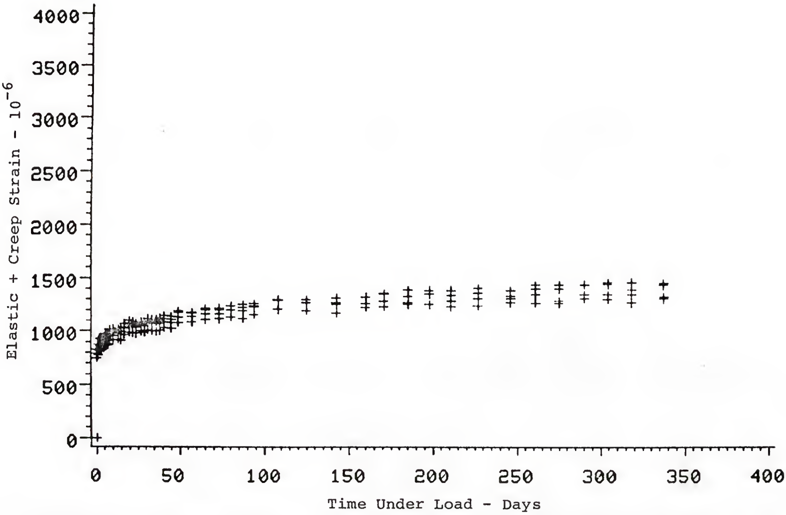


Figure C.4. Elastic Plus Creep Strain Response for Miami Oolite Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Moist Room.

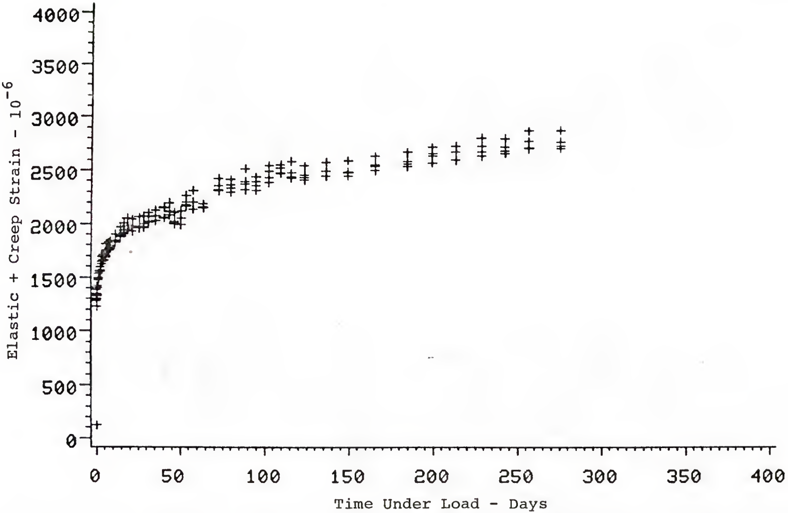


Figure C.5. Elastic Plus Creep Strain Response for Miami Oolite Loaded at 90 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

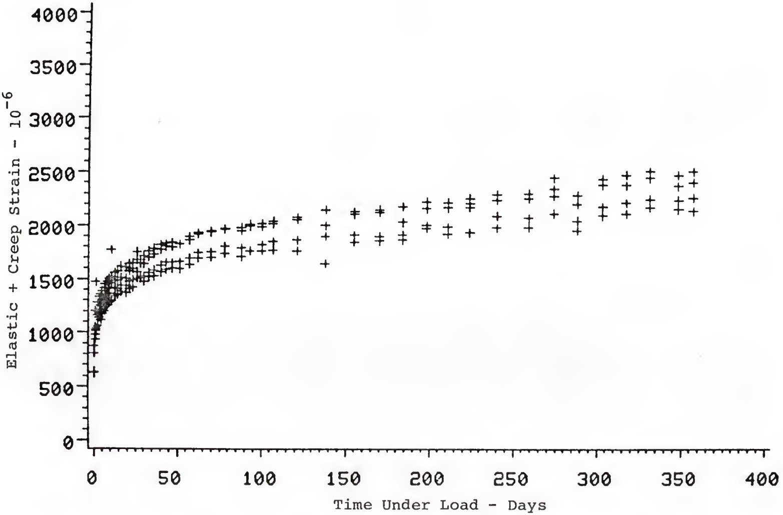


Figure C.6. Elastic Plus Creep Strain Response for Brooksville Loaded at 7 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

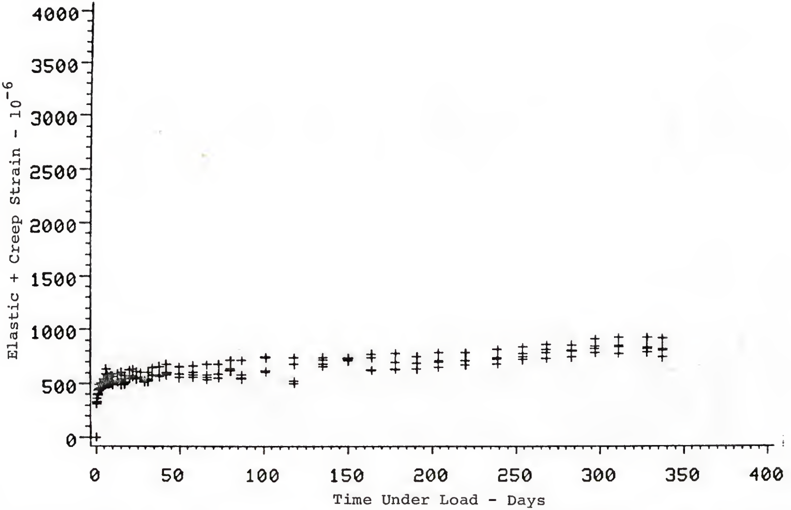


Figure C.7. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

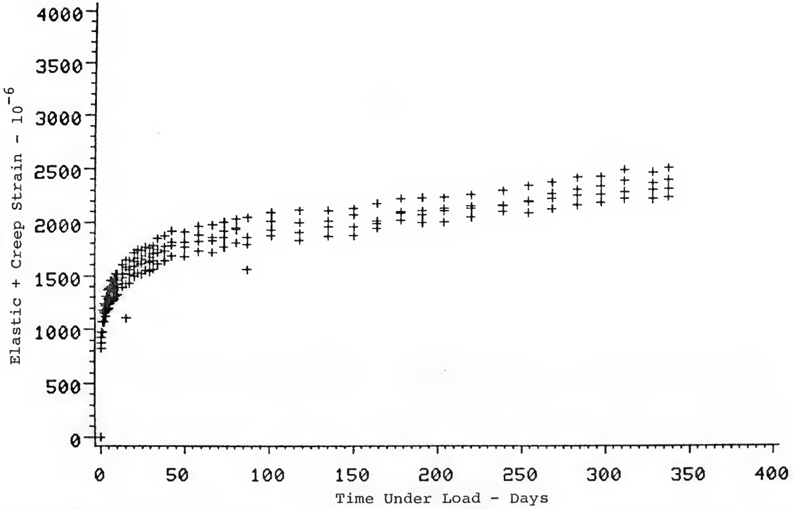


Figure C.8. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 50 Percent f_c' and Stored in the Air Conditioned Room.

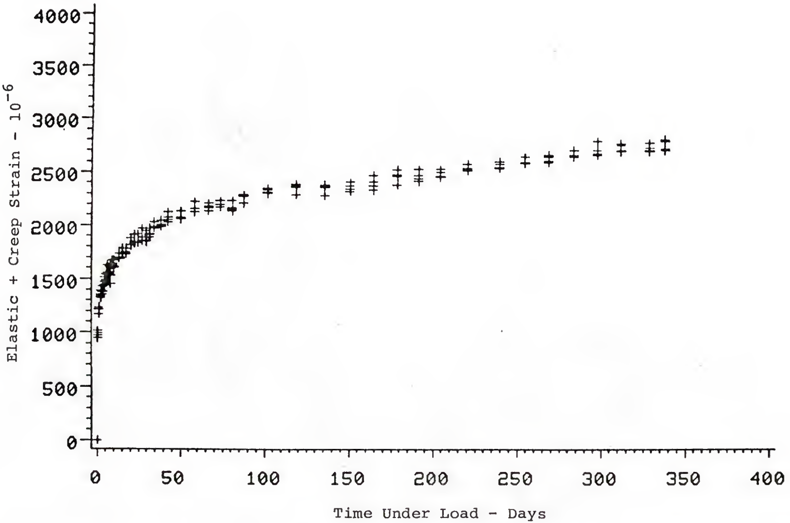


Figure C.9. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

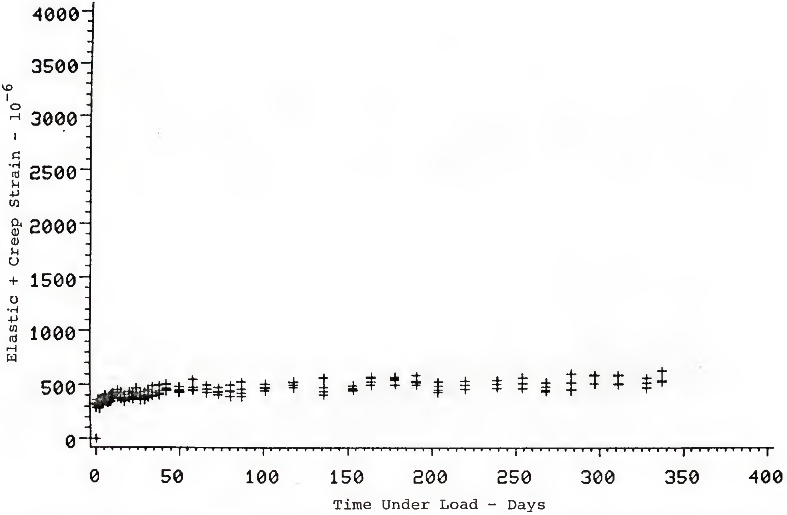


Figure C.10. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 25 Percent f_c' and Stored in the Moist Room.

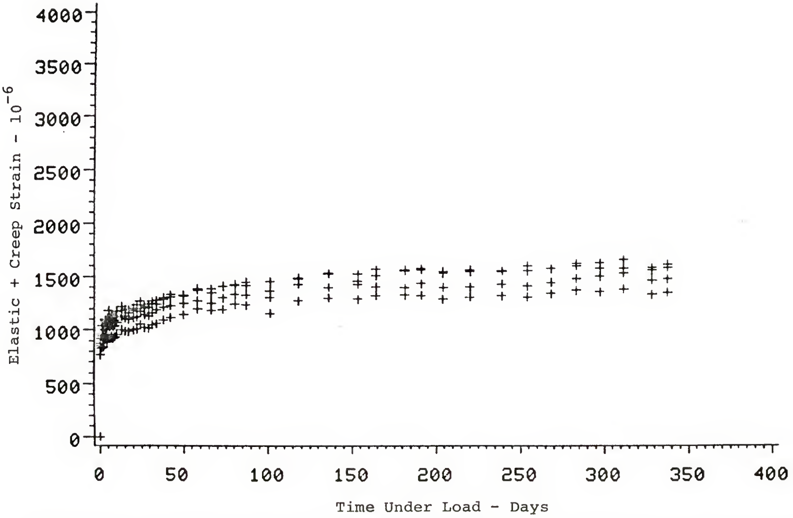


Figure C.11. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Moist Room.

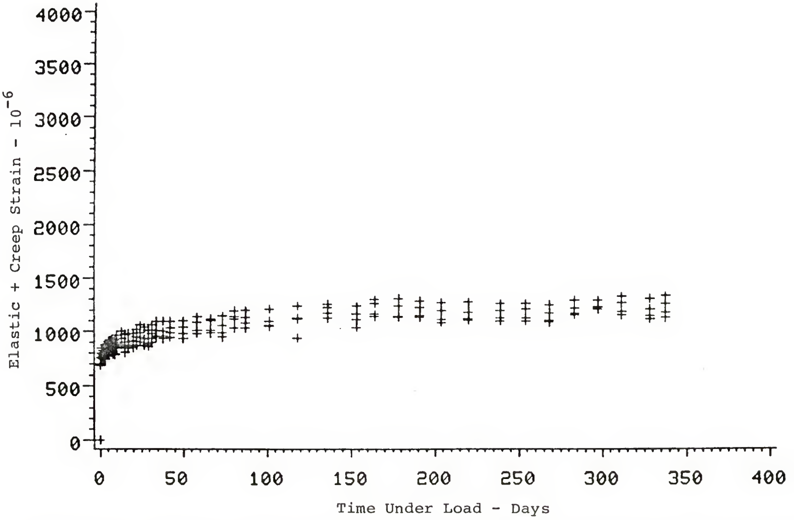


Figure C.12. Elastic Plus Creep Strain Response for Brooksville Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Moist Room.

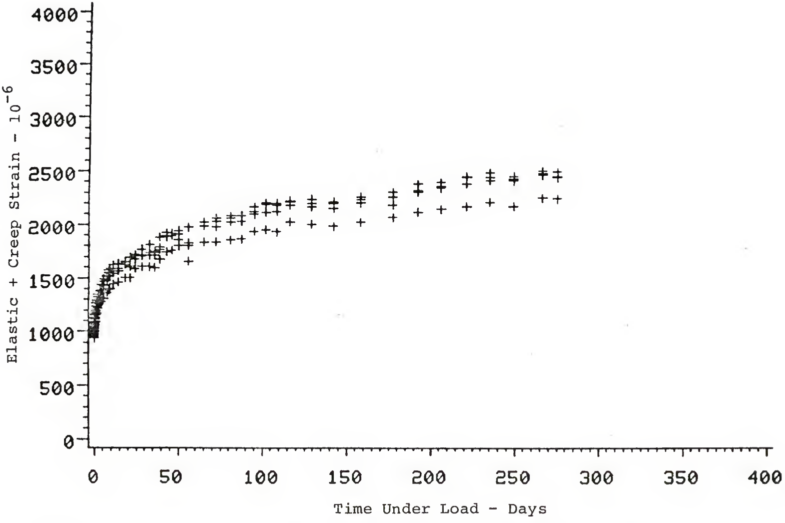


Figure C.13. Elastic Plus Creep Strain Response for Brooksville Loaded at 90 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

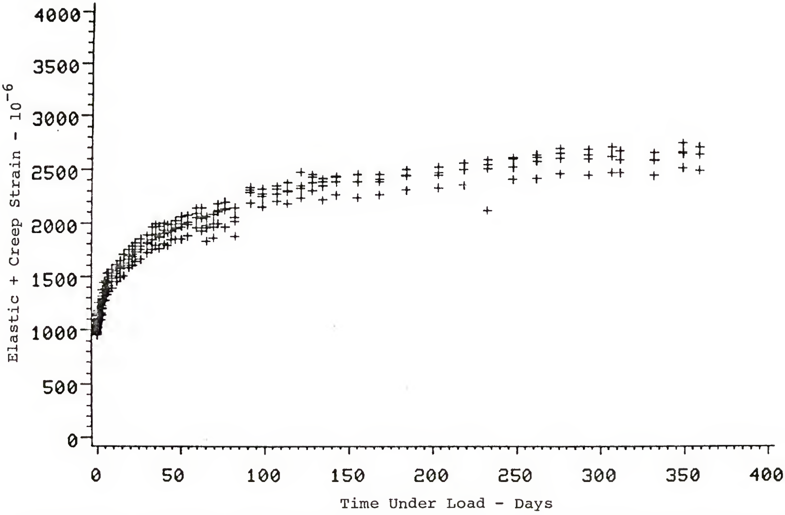


Figure C.14. Elastic Plus Creep Strain Response for Ft. Thompson Loaded at 7 Days to Stress of 50 Percent f_c' and Stored in the Air Conditioned Room.

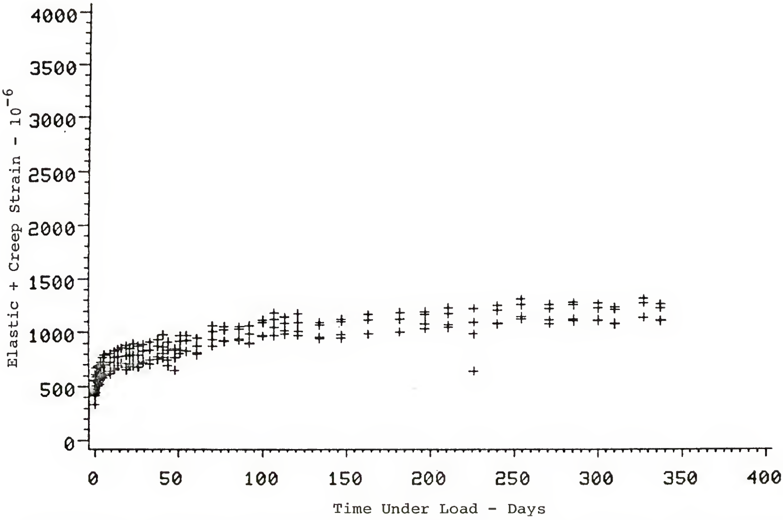


Figure C.15. Elastic Plus Creep Strain Response for Ft. Thompson Loaded at 28 Days to Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

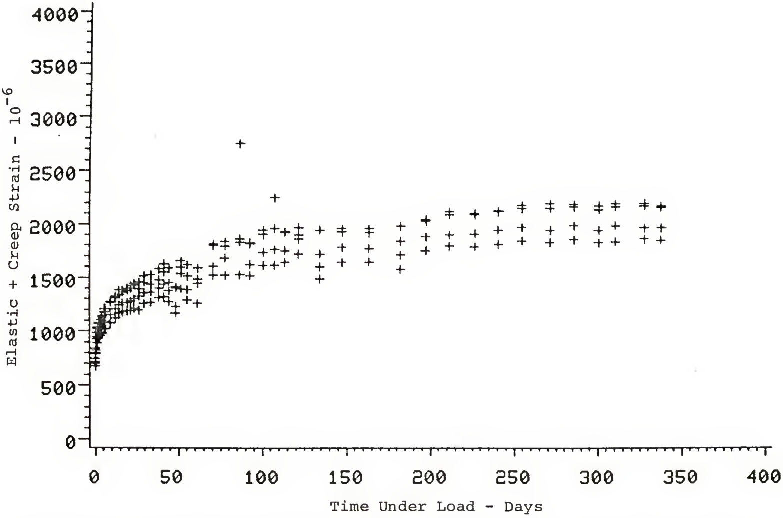


Figure C.16. Elastic Plus Creep Strain Response for Ft. Thompson Loaded at 28 Days to Stress of 40 Percent f'_c and Stored in the Air Conditioned Room.

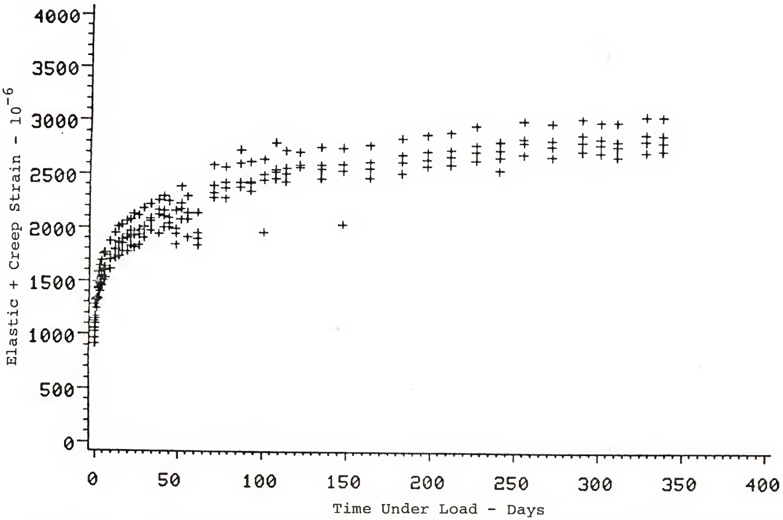


Figure C.17. Elastic Plus Creep Strain Response for Ft. Thompson Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

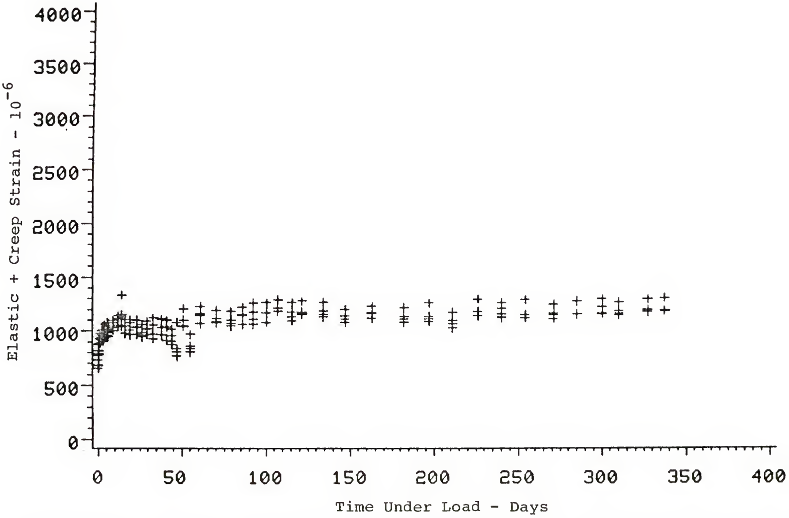


Figure C.18. Elastic Plus Creep Strain Response for Ft. Thompson Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Moist Room.

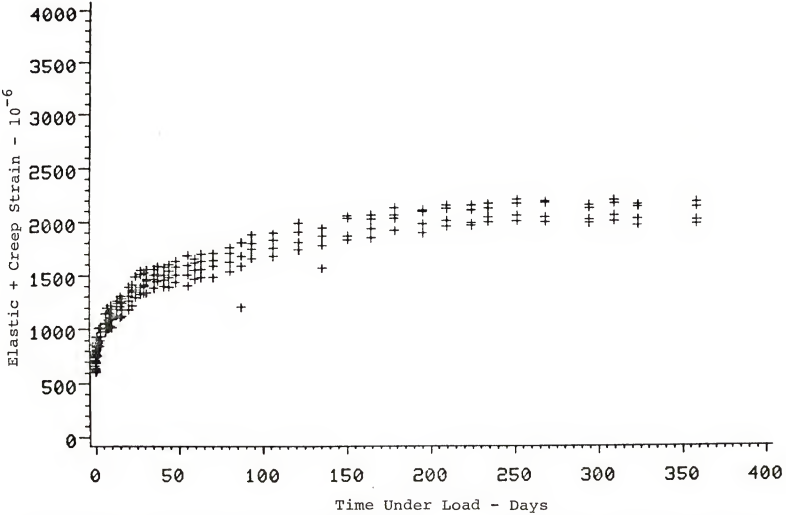


Figure C.19. Elastic Plus Creep Strain Response for Calera Loaded at 7 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

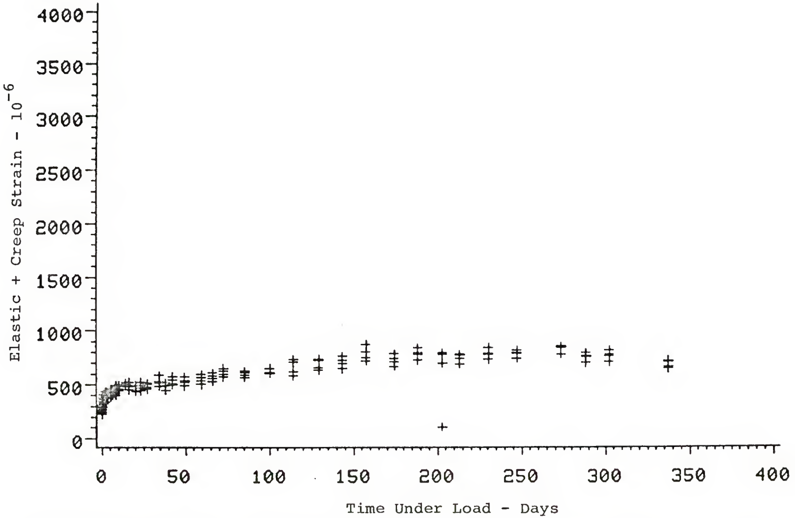


Figure C.20. Elastic Plus Creep Strain Response for Calera Loaded at 28 Days to Stress of 25 Percent f'_c and Stored in the Air Conditioned Room.

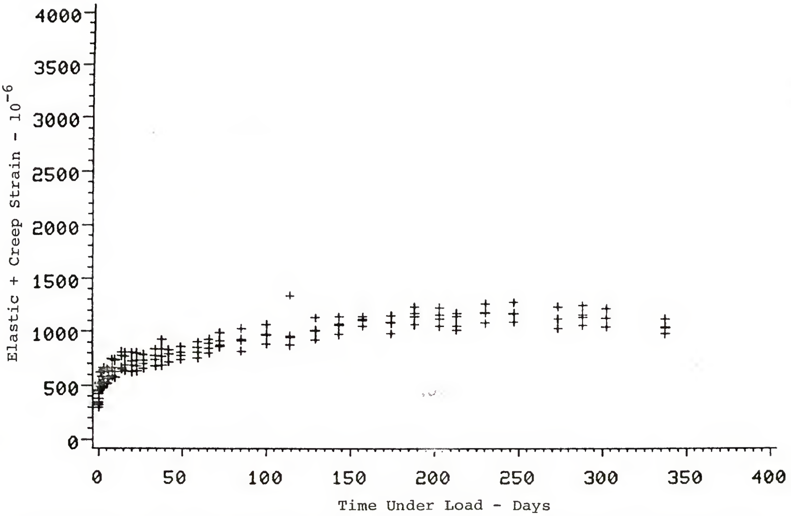


Figure C.21. Elastic Plus Creep Strain Response for Calera Loaded at 28 Days to Stress of 33 Percent f'_c and Stored in the Air Conditioned Room.

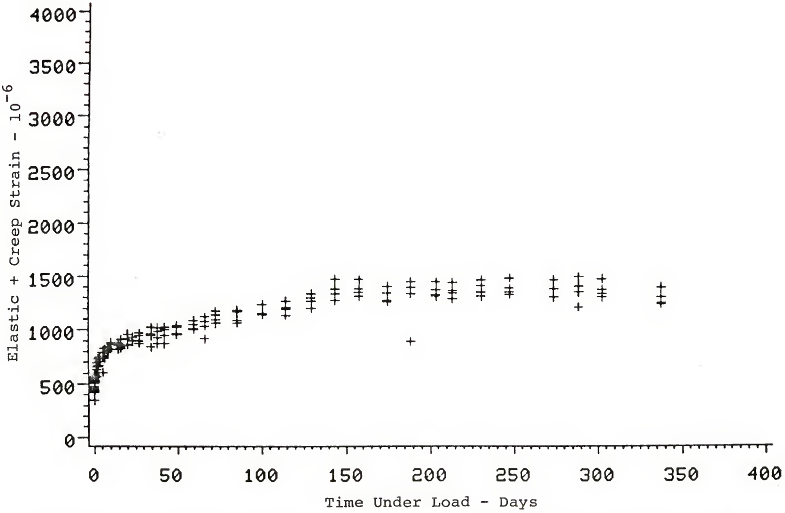


Figure C.22. Elastic Plus Creep Strain Response for Calera Loaded at 28 Days to Stress of 40 Percent f'_c and Stored in the Air Conditioned Room.

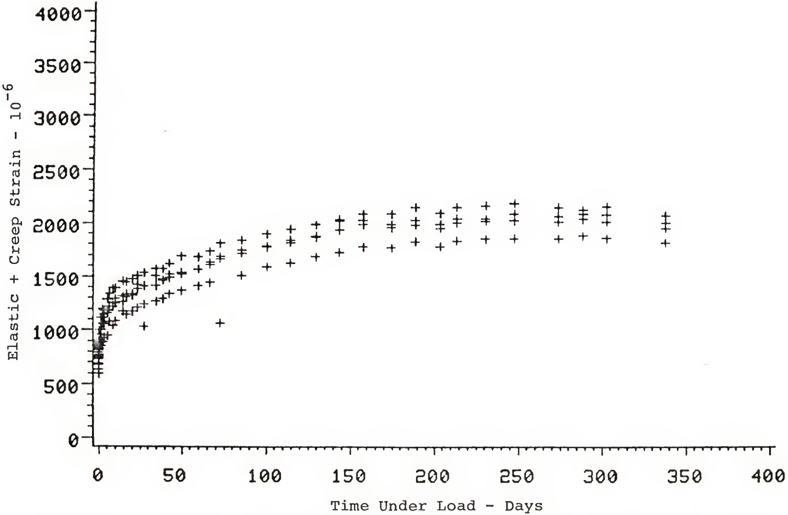


Figure C.23. Elastic Plus Creep Strain Response for Calera Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Air Conditioned Room.

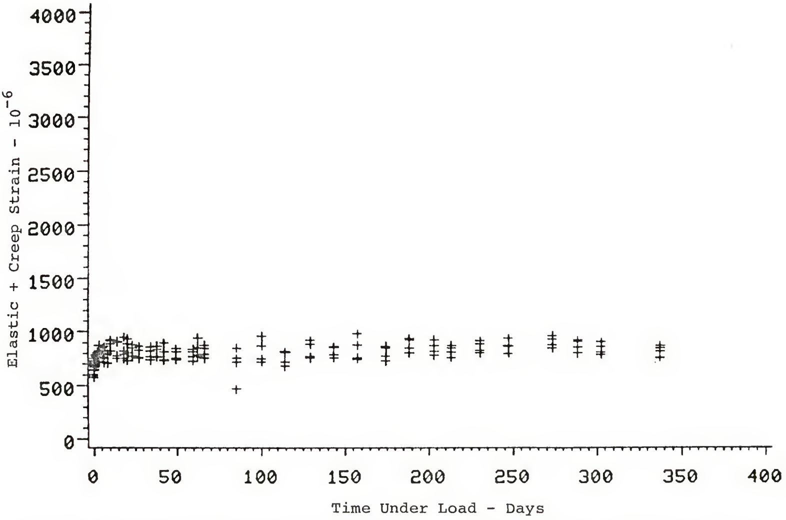


Figure C.24. Elastic Plus Creep Strain Response for Calera Loaded at 28 Days to Stress of 50 Percent f'_c and Stored in the Moist Room.

APPENDIX D

COMPRESSIVE STRENGTH DATA, TENSILE STRENGTH DATA, MODULUS OF ELASTICITY DATA, AND MODEL COEFFICIENTS

Table D.1. Compressive Strength (f'_c) of Miami Oolite Concrete.

Age of Concrete (Days)	Storage	Cylinder f'_c (psi) 1	2	3	4	Mean f'_c (psi)	Standard Deviation
3	A/C	3764	3693	3226	3194	3469	301
3	M/R	3502	3488	3523	3538	3513	22
14	A/C	5153	4782	4574	5203	4924	297
14	M/R	4740	4775	4425	---	4647	193
28	A/C	5543	5218	5416	5182	5340	170
28	M/R	5044	5504	4882	5182	5153	264
60	A/C	5482	5186	5532	5723	5481	222
60	M/R	5522	4676	5271	5939	5352	528
90	A/C	5716	5738	5621	6335	5853	326
90	M/R	6067	5812	6445	5883	6052	283
180	A/C	4996	5087	6201	5585	5442	583
180	M/R	5684	6084	6062	5649	5870	235
268	A/C	5270	5571	5847	5684	5593	243
268	M/R	6211	6105	6180	5713	6052	231
365	A/C	5246	5713	5398	6095	5613	376
365	M/R	5904	5348	5557	6010[1]	5705	307

[1] A fifth cylinder was tested at 6410 psi.

Table D.2. Compressive Strength (f'_c) of Brooksville Concrete.

Age of Concrete (Days)	Storage	Cylinder f'_c (psi) 1	2	3	4	Mean f'_c (psi)	Standard Deviation (psi)
3	A/C	2144	2080	2186	---	2137	53
3	M/R	2345	2260	2140	---	2249	103
7	A/C	3251	3254	3102	---	3201	87
7	M/R	2932	3258	---	---	3095	231
28	A/C	4216	3622	3866	3820	3880	247
28	M/R	3937	3944	4018	---	3966	45
59	A/C	3962	3689	3852	3863	3842	113
59	M/R	4255	4481	---	---	4372	160
90	A/C	4185	4022	---	---	4103	116
90	M/R	4316	4570	4220	---	4369	181
198	A/C	3527	3615	3877	---	3673	182
198	M/R	4722	4404	4280	---	4469	228
267	A/C	3926	3813	3873	3389	3750	245
267	M/R	4973	4280	4305	4758	4579	342
365	A/C	3965	3809	3813	4004	3898	101
365	M/R	4789	4167	4372	4563	4473	266

Table D.3. Compressive Strength (f'_c) of Ft. Thompson Concrete.

Age of Concrete (Days)	Storage	Cylinder f'_c (psi) 1	2	3	4	Mean (f'_c) (psi)	Standard Deviation
7	A/C	4524	4595	4531	4698	4587	81
7	M/R	4453	4662	4170	4386	4418	203
14	A/C	5430	5196	4786	4942	5088	284
14	M/R	4690	4973	4956	4609	4807	160
28	A/C	5005	5401	5469	5295	5293	205
28	M/R	5359	5140	5069	5246	5203	127
61	A/C	5207	5034	5027	5380	5162	167
61	M/R	5033	4899	4970	5051	4989	69
91	A/C	5391	5684	5030	4821	5232	383
91	M/R	5324	5575	5677	5745	5580	160
180	A/C	5596	4754	5207	4821	5095	389
180	M/R	6035	5957	5851	5465	5827	219
273	A/C	4428	4690	4771	4990	4720	232
273	M/R	5677	5648	5853	4492	5417	540
365[1]	A/C	5415	5051	5539	5284	5372	212
365[2]	M/R	6083	5666	6076	6076	5999	186

[1] A fifth cylinder was tested at 5570 psi f'_c

[2] A fifth cylinder was tested at 6093 psi f'_c

Table D.4. Compressive Strength (f'_c) of Calera Concrete.

Age of Concrete (Days)	Storage	Cylinder f'_c (psi)				Mean (f'_c) (psi)	Standard Deviation (psi)
		1	2	3	4		
7	A/C	4556	4482	4383	4694	4529	131
7	M/R	4644	4464	4648	4899	4664	179
14	A/C	4351	5041	5062	3948	4600	546
14	M/R	5016	5444	4825	4708	4998	323
28	A/C	4705	4613	5239	4772	4832	279
28	M/R	6222	5430	5111	5186	5487	440
62	A/C	4995	5200	4931	4935	5015	127
62	M/R	5133	5989	6200	5299	5655	519
90	A/C	5119	4655	4910	5009	4923	198
90	M/R	6151	5692	5730	6657	6058	451
182	A/C	4736	4916	4997	---	4883	134
182	M/R	6554	7166	5832	---	6517	668
272	A/C	4534	4845	4690	5217	4822	293
272	M/R	7013	7360	6967	6794	7033	237
365	A/C	4916	4527	4534	4874	4713	211
365	M/R	6384	7629	6614	7059	6921	549

Table D.5. Equations Fitted to Compressive Strength Data.

Equation	Storage	Concrete	A	B	R^2
$t = Ae^{By}$	A/C	Miami Oolite	4008.3	0.00106	0.625
		Brooksville	2766.7	0.00104	0.552
		Ft. Thompson	4845.9	0.000193	0.080
		Calera	4595.6	0.000176	0.083
	M/R	Miami Oolite	3770.0	0.00139	0.798
		Brooksville	2590.0	0.00175	0.837
		Ft. Thompson	4201.5	0.000919	0.682
		Calera	4013.4	0.00156	0.799
$y = Ae^{Bt}$	A/C	Miami Oolite	4675.1	0.000890	0.228
		Brooksville	3206.7	0.000898	0.153
		Ft. Thompson	4988.5	0.000241	0.477
		Calera	4564.2	0.001086	0.243
	M/R	Miami Oolite	4532.3	0.00135	0.458
		Brooksville	3186.3	0.00165	0.438
		Ft. Thompson	4697.6	0.00134	0.672
		Calera	4786.2	0.00281	0.589
$y = Ae^{-B/t}$	A/C	Miami Oolite	5625.1	1.472	0.872
		Brooksville	3917.9	1.813	0.921
		Ft. Thompson	5271.1	0.846	0.385
		Calera	4970.0	0.746	0.276
	M/R	Miami Oolite	5738.8	1.535	0.845
		Brooksville	4423.6	2.124	0.925
		Ft. Thompson	5539.7	1.699	0.710
		Calera	5931.3	1.842	0.613

Table D.5. (Cont'd.)

Equation	Storage	Concrete	A	B	R^2
$y = At^B$	A/C	Miami Oolite	3522.8	0.0969	0.656
		Brooksville	2335.5	0.1041	0.578
		Ft. Thompson	4610.0	0.0261	0.195
		Calera	4175.5	0.0403	0.300
	M/R	Miami Oolite	3278.4	0.1189	0.862
		Brooksville	2121.8	0.1506	0.859
		Ft. Thompson	3858.3	0.0777	0.788
		Calera	3858.4	0.0990	0.664

Table D.6. Tensile Strength (f_t) of Concretes Stored in the Air Conditioned Room.

Concrete	Age of Concrete (Days)	Cylinder f_t (psi)				Mean f_t (psi)	Standard Deviation (psi)
		1	2	3	4		
Miami Oolite	3	389.93	329.80	315.66	176.84	303.10	90.10
	28	442.10	376.67	309.47	404.96	383.30	56.04
	90	525.21	386.39	463.32	439.44	469.51	57.56
Brooksville	3	270.56	243.15	229.89	247.57	247.80	16.94
	7	362.52	335.11	328.04	310.35	334.23	21.68
	28	286.48	244.04	259.95	247.57	259.51	19.23
	90	329.80	353.68	257.30	247.57	298.86	50.48
Ft. Thompson	7	418.22	381.97	408.50	443.87	413.18	25.58
	28	453.59	401.42	383.74	315.66	388.60	56.96
Calera	7	384.62	392.58	419.99	369.59	391.70	21.14
	14	565.55	580.92	427.07	434.14	496.92	77.98
	28	339.53	351.03	325.38	300.63	329.10	21.71

Table D.7. Modulus of Elasticity.

Concrete	Storage	Age of Concrete (Days)	Mean Modulus of Elasticity (10^6 psi)
Miami Oolite	A/C	14	3.665
		28	3.675
		59	3.344
		60	3.760
		198	3.563
		267	3.944
		365	4.024
	M/R	14	3.789
		28	3.684
		59	3.964
		60	4.431
		198	4.021
		267	4.224
		365	4.329
Brooksville	A/C	7	2.562
		28	2.714
		59	3.045
		90	2.867
		198	2.723
		267	3.150
		365	3.198
	M/R	7	2.774
		28	2.477
		59	3.291
		90	3.359
		198	3.139
		267	3.775
		365	3.741
Ft. Thompson	A/C	7	3.575
		14	3.513
		28	3.775
		61	3.732
		91	3.732
		180	3.495
		273	4.091
		365	4.063

Table D.7. Cont'd.

Concrete	Storage	Age of Concrete (Days)	Mean Modulus of Elasticity (10^6 psi)
Ft. Thompson	M/R	7	3.520
		14	3.859
		28	3.690
		61	3.888
		91	4.140
		180	4.241
		273	4.548
		365	4.753
Calera	A/C	7	4.470
		14	4.648
		20	4.513
		62	4.123
		90	4.462
		182	4.516
		272	4.334
		365	4.421
	M/R	7	3.969
		14	5.624
		28	5.943
		62	6.002
		90	4.191
		182	6.612
		272	6.898
		365	7.034

Table D.8. Predicted Elastic Strains Compared to Actual Elastic Strains.

Concrete	Storage	% f'_c	Age of Loading	Experimental $E \times 10^6$ psi	Predicted $\varepsilon \times 10^{-6}$	Actual $\varepsilon \times 10^{-6}$	% Diff.
Miami Oolite	A/C	50	3	---	---	791.22	---
		25	28	3.675	363.21	454.75	25.2
		50	28	3.675	726.41	936.61	28.9
	M/R	50	20	3.684	699.29	790.25	13.0
	A/C	50	90	3.760	778.14	1158.22	48.8
Brooksville	A/C	50	7	2.562	624.66	737.25	18.0
		25	28	2.714	357.46	338.03	-5.44
		50	28	2.714	714.92	902.86	26.3
		50	28	2.714	714.92	984.53	37.7
	M/R	25	28	2.977	333.90	315.45	-5.53
		50	28	2.977	667.81	852.54	27.7
		50	28	2.977	667.81	709.23	6.20
	A/C	50	90	2.867	713.40	968.01	35.7
Ft. Thompson	A/C	50	7	3.575	641.46	1014.82	58.2
		25	28	3.775	350.41	410.07	17.0
		40	28	3.775	560.73	754.54	34.6
		50	28	3.775	700.80	1005.58	43.5
	M/R	50	28	3.690	716.94	838.34	16.9
Calera	A/C	50	7	4.470	506.46	671.21	32.5
		25	28	4.513	267.63	251.08	-6.18
		33	28	4.513	353.27	379.14	7.32
		40	28	4.513	428.21	473.43	10.6
		50	28	4.513	535.26	733.16	37.0
	M/R	50	28	5.943	406.46	616.46	51.7

Table D.9. ACI Committee 209 Creep Model Coefficients.

Cu = Ultimate Creep Coefficient = 2.35

Ka:

Age of Concrete	Ka
3	1.423
7	1.573
28	1.852
90	2.126

K_h: 100% Relative Humidity $K_h = 0.60$

60% Relative Humidity $K_h = 0.87$

K_{th}: $K_{th} = 1$

K_s:

Concrete	Slump (inches)	K_s
Calera	1	0.887
Brooksville	4	1.09
Miami Oolite	2.75	1.00
Ft. Thompson	2	0.95

K_f:

Concrete	% Fines	K_f
Miami Oolite	33.5	0.960
Brooksville	33.5	0.960
Ft. Thompson	30.2	0.952
Control	27.8	0.947

K_e: 4% Air $K_e = 0.82$ Use 1.0

Table D.10. C.E.B. Creep Model Coefficients.

$$\phi_t = K_c\ K_d\ K_b\ K_e\ K_t$$

K_c: 100% Relative Humidity $K_c = 1$

60% Relative Humidity $K_c = 2.7$

K_d:

Age at Loading	K_d
3	1.6
7	1.4
28	1.0
90	0.75

$K_b = K_b = 0.82$ dm = 7.62 cm

K_e: $K_e = 1.1$

K_t:

Days	K_t
10	0.28
20	0.40
40	0.53
60	0.60
100	0.70
200	0.81
300	0.87
400	0.90

Table D.11. Bazant-Panula Simplified Creep Model Coefficients.

τ_{sh}: $D = 76.2$ $C_1(t_o) = 122.4$

$K_s = 1.15$ $\tau_{sh} = 62.737$

Basic Creep: $\alpha = 0.05$

$$C_o(t,t') = \frac{\phi_1}{E_o}(t^{1-m} + \alpha)(t - t')^n$$

$$\phi_1 = 0.3 + 15(f_c')^{-1.2}$$

$$m = 0.28 + 1/(f_c')^2$$

$$n = 0.115 + 0.002\ (f_c')^3$$

where f_c' = 28 day compressive strength

Drying Creep:

Concrete	FC	Z	$E_{sh}00\times10^{-6}$	r	ψ_d
Miami Oolite	5.34	0.808	1228	0.2088	0.008177
Brooksville	3.88	0.808	1329	-0.284	0.00560
Ft. Thompson	5.29	2.651	1221	-0.169	0.00560
Calera	4.83	3.333	1097	0.048	0.00598

Table D.12. ACI Shrinkage Model Coefficients.

$\varepsilon_{sh} = \varepsilon_{shu}\ S_t\ S_n\ S_{th}\ S_s\ S_f\ S_e\ S_c$

Mean $\varepsilon_{shu} = 0.00080$

S_t: $S_t = \frac{t}{35} + t$

S_h: h = 60% Relative Humidity - $S_h = 0.80$

h = 100% Relative Humidity - $S_h = 1.0$

S_{th}: $S_{th} = 1$

S_s:

Concrete	Slump(s)	S_s
Miami Oolite	2.75	1.002
Brooksville	4.00	1.054
Ft. Thompson	2.00	0.972
Calera	1.00	0.943

S_f:

Concrete	S/a	S_f
Miami Oolite	0.449	0.929
Brooksville	0.439	0.915
Ft. Thompson	0.446	0.925
Calera	0.415	0.881

S_e: $S_e = 0.98$ for 4% Air

S_c: $S_c = 0.987$

Table D.13. C.E.B. Shrinkage Model Coefficients.

$$\varepsilon_{sh} = \varepsilon_c K_b K_e K_p K_t$$

where: $\varepsilon_c = 33 \times 10^{-5}$

$K_b = 0.82$

$K_e = 1.1$

$K_p = 1.0$

K_t = same as Creep--Table D.8.

Table D.14. Bazant-Panula Simplified Creep Model Coefficients.

K_h = 0.784 for 60% Relative Humidity

K_h = 1.000 for 100% Relative Humidity

τ_{sh} = 62.737

Concrete	$E_{sh\alpha}$ (10^{-6})
Miami Oolite	1228
Brooksville	1329
Ft. Thompson	1221
Calera	1097

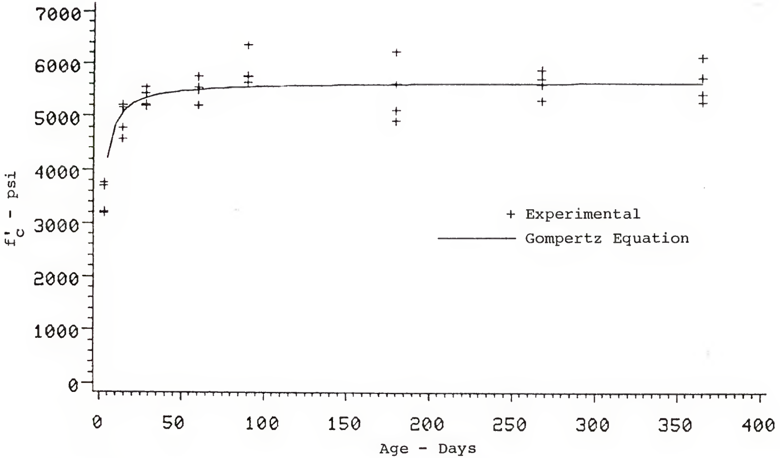


Figure D.1. Comparison of Compressive Strength of Miami Oolite Concrete Stored in the Air Conditioned Room and the Fitted Gompertz Equation.

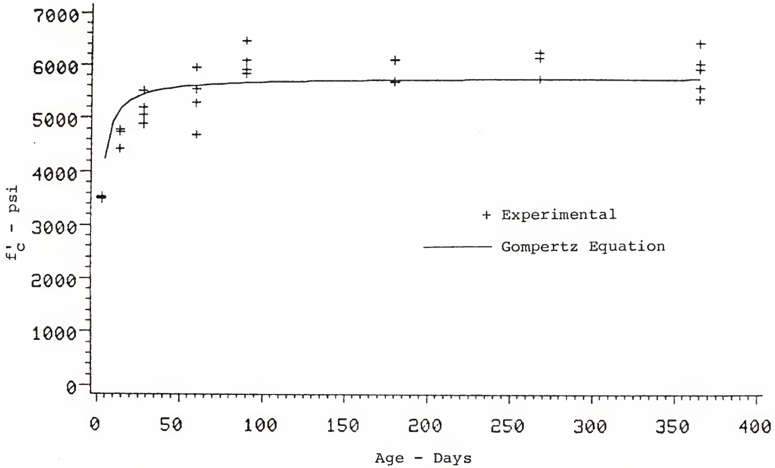


Figure D.2. Comparison of Compressive Strength of Miami Oolite Concrete Stored in the Moist Room and the Fitted Compertz Equation.

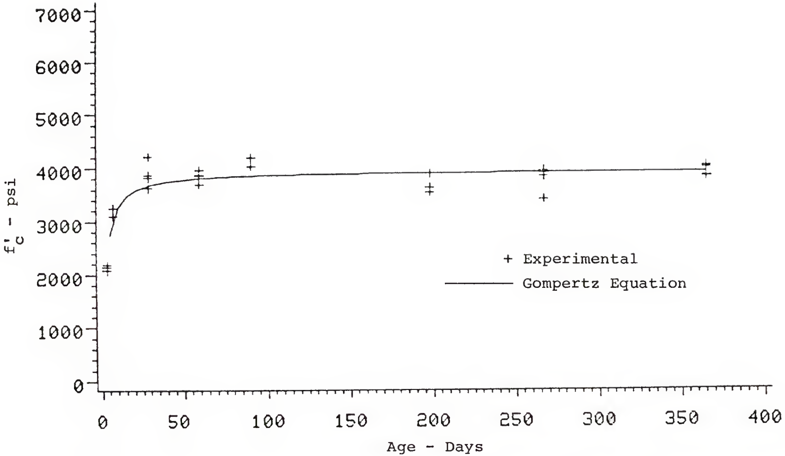


Figure D.3. Comparison of Compressive Strength of Brooksville Concrete Stored in the Air Conditioned Room and the Fitted Gompertz Equation.

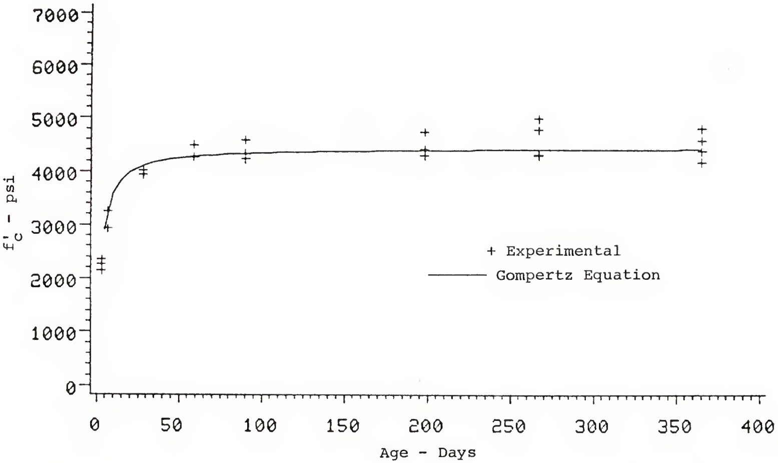


Figure D.4. Comparison of Compressive Strength of Brooksville Concrete Stored in the Moist Room and the Fitted Gompertz Equation.

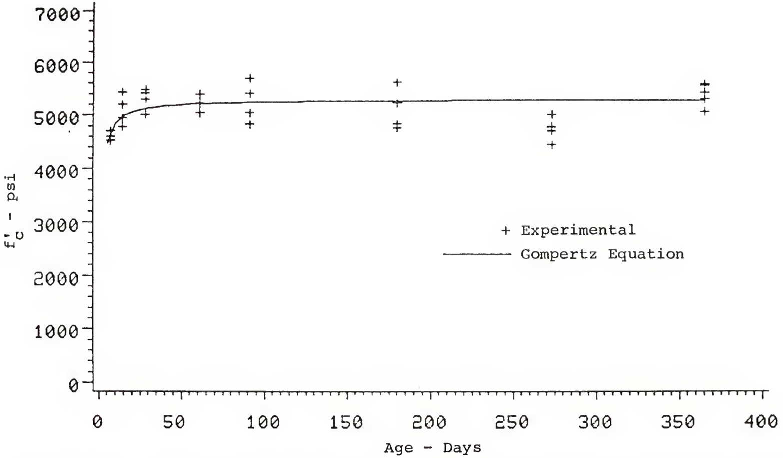


Figure D.5. Comparison of Compressive Strength of Ft. Thompson Concrete Stored in the Air Conditioned Room and the Fitted Gompertz Equation.

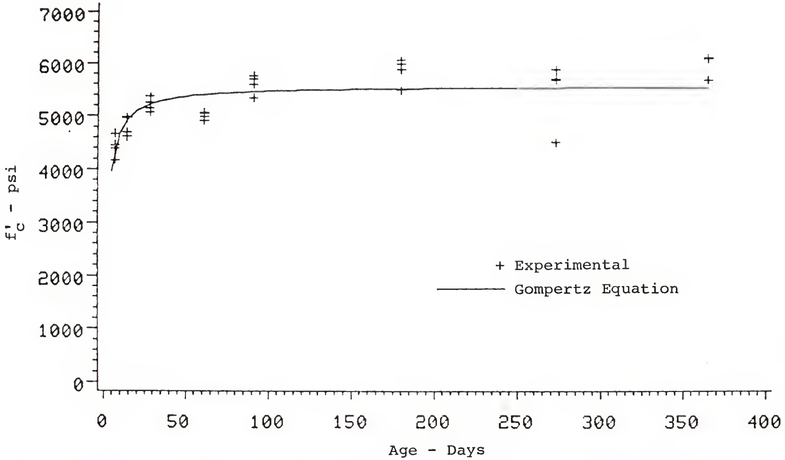


Figure D.6. Comparison of Compressive Strength of Ft. Thompson Concrete Stored in the Moist Room and the Fitted Gompertz Equation.

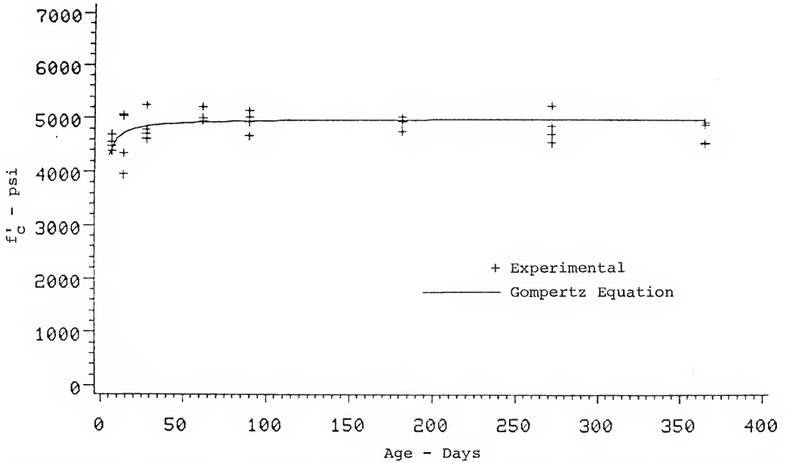


Figure D.7. Comparison of Compressive Strength of Calera Concrete Stored in the Air Conditioned Room and the Fitted Gompertz Equation.

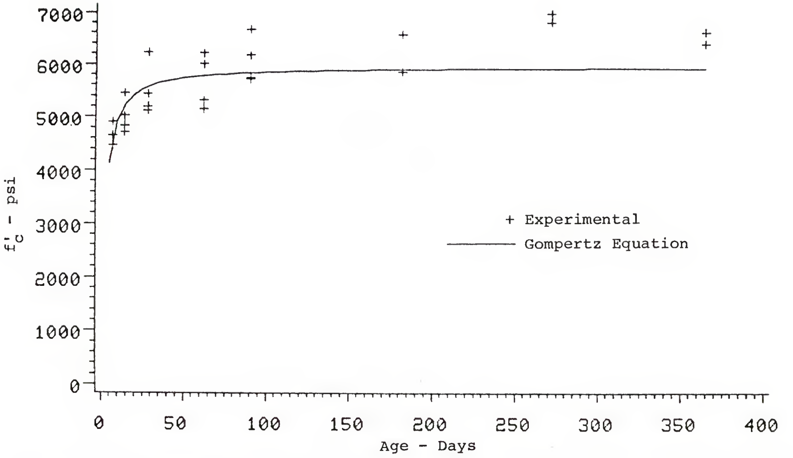


Figure D.8. Comparison of Compressive Strength of Calera Concrete Stored in the Moist Room and the Fitted Gompertz Equation.

BIBLIOGRAPHY

1. ACI Committee 209, "Prediction of Creep," Designing for Creep & Shrinkage in Concrete, ACI Publication SP-76, American Concrete Institute, Detroit, 1982.

2. ASTM C39-72, "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens," Book of Standards, American Society for Testing and Materials, Part 14,Philadelphia, 1984.

3. ASTM C138-74, "Standard Method of Test for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete," Book of Standards, American Society for Testing and Materials, Part 14, Philadelphia, 1984.

4. ASTM C143-71, "Standard Method of Test for Slump of Portland Cement Concrete," Book of Standards, American Society for Testing and Materials, Part 14, Philadelphia, 1984

5. ASTM C173-73, "Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method," Book of Standards, American Society for Testing and Materials, Part 14, Philadelphia, 1984.

6. ASTM C469-65, "Standard Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression," Book of Standards, American Society for Testing and Materials, Part 14, Philadelphia, 1984.

7. ASTM C496-71, "Standard Method of Test for Splitting Tensile Strength of Cylindrical Concrete Specimens, Book of Standards, American Society for Testing and Materials, Part 14,Philadelphia, 1984.

8. ASTM C511-73, "Standard Specification for Moist Cabinets and Rooms Used in the Testing of Hydraulic Cements and Concretes," Book of Standards, American Society for Testing and Materials, Part 14, Philadelphia, 1984.

9. Bazant, Z. P., "Constitutive Equation for Concrete Creep and Shrinkage Based on Thermodynamics of Multiphase Systems," Materials and Structures, No. 13, January-February, 1970, pp. 3-36.

10. Bazant, Z. P., "Prediction of Concrete Creep Effects Using Age-Adjusted Effective Modulus Method," Journal American Concrete Institute, Vol. 69, No. 4, April 1972, pp. 212-217.

11. Bazant, Z. P., "Viscoelasticity of Solidifying Porous Material--Concrete," Journal of Engineering Mechanics Division, ASCE, Vol. 103, No. EM6, December 1977, pp. 1049-1067.

12. Bazant, A. P., "Mathematical Models for Creep and Shrinkage of Concrete," Chapter 7, Creep and Shrinkage in Concrete Structures, John Wiley & Sons, New York, 1982, pp. 163-256.

13. Bazant, Z. P. and A. A. Asghari, "Constitutive Law for Nonlinear Creep of Concrete," Journal of Engineering Mechanics Division, ASCE, Vol. 103, No. EM1, February 1977, pp. 113-124.

14. Bazant, Z. P. and S. S. Kim, "Nonlinear Creep of Concrete--Adaptation and Flow," Journal of Engineering Mechanics Division, ASCE, Vol. 105, No. EM3, June 1979, pp. 429-446.

15. Bazant, Z. P. and S. S. Kim, "Approximate Relaxation Function for Concrete," Journal of the Structural Division, ASCE, Vol. 105, No. ST12, December 1979, pp. 2695-2705.

16. Bazant, Z. P. and L. J. Najjar, "Comparison of Approximent Linear Methods for Concrete Creep," Journal of the Structural Division, ASCE, Vol. 99, No. ST9, September 1973, pp. 1851-1874.

17. Bazant, Z. P. and Liisa Panula, "Creep and Shrinkage Characterization for Analyzing Prestressed Concrete Structure," Journal of the Prestressed Concrete Institute, Vol. 25, No. 3, May/June 1980, pp. 86-122.

18. Bazant, Z. P., T. Tsubak, and Z. Celep, "Singular History Integral for Creep Rate of Concrete," Journal of Engineering Mechanics Division, ASCE, Vol. 109, No. 3, June 1983, pp. 866-884.

19. Bazant, Z. P. and S. T. Wu, "Creep and Shrinkage Law for Concrete at Variable Humidity," Journal of Engineering Mechanics Division, ASCE, Vol. 100, No. EM6, December 1974, pp. 1183-1209.

20. Bennett, E. W. and D. R. Loat, "Shrinkage and Creep of Concrete as Affected by the Fineness of Portland Cement," Magazine of Concrete Research, Vol. 22, No. 71, June 1970, pp. 69-78.

21. Brooks, J. J. and P. J. Wainwright, "Properties of Ultra-High-Strength Concrete Containing a Superplasticizer," Magazine of Concrete Research, Vol. 35, No. 125, December 1983, pp. 205-213.

22. Brooks, J. J., P. J. Wainwright, and A. M. Neville, "Superplasticizer Effect on Time-Dependent Properties of Air Entrained Concrete," Concrete, Vol. 13:6, June 1979, pp. 35-38.

23. Brooks, J. J., P. J. Wainwright, and A. M. Neville, "Time-Dependent Behavior of High-Early-Strength Concrete Containing Superplasticizer," Developments in the Use of Superplasticizers, ACI SP-68, American Concrete Institute, Detroit, 1981.

24. Buettner, D. R. and R. L. Hollrah, "Creep of Preshrunk Concrete," Journal American Concrete Institute, Vol. 66, No. 9, September 1969, pp. 737-740.

25. Comite Europeen du Beton (CEB), International Recommendations for the Design and Construction of Concrete Structures, Cement and Concrete Association, London, August 1978.

26. Dhir, R. K. and A. W. F. Yap, "Superplasticized Flowing Concrete: Strength and Deformation Properties," Magazine of Concrete Research, Vol. 35, No. 125, September 1983, pp. 100-200.

27. Dhir, R. K. and A. W. F. Yap, "Superplasticized High-Workability Concrete: Some Properties in the Fresh and Hardened States," Magazine of Concrete Research, Vol. 35, No. 125, December 1983, pp. 214-228.

28. Gamble, W. L., "Creep of Concrete in Variable Environments," Journal of the Structural Division, ASCE, Vol. 108, No. ST10, October 1982, pp. 2211-2222.

29. Gamble, B. R. and L. J. Parrott, "Creep of Concrete in Compression During Drying and Wetting," Magazine of Concrete Research, Vol. 30, No. 104, September 1978, pp. 129-138.

30. Glucklich, "Creep Mechanism in Cement Mortar," Journal American Concrete Institute, Vol. 59, July 1962, pp. 923-948.

31. Hamada, H. S., S. Zundelevich, and A. N. L. Chiu, "Time-Dependent Behavior of Concrete Made with Hawaiian Aggregates," Highway Research Record, No. 400, 1972, pp. 37-54.

32. Jordaan, I. J., G. L. England, and M. A. Khalifa, "Creep of Concrete: A Consistent Engineering Approach," Journal of the Structural Division, ASCE, Vol. 103, No. ST3, March 1977, pp. 475-491.

33. Li, S.-T. and V. Ramakrishnan, "Young's Modulus, Creep, and Shrinkage of Gap-Graded Concrete Versus Continuously Graded Concrete," Highway Research Record, No. 324, 1970, pp. 77-88.

34. Lyse, I. L., "The Shrinkage and Creep of Concrete," Magazine of Concrete Research, Vol. 11, No. 33, November 1959, pp. 143-150.

35. Madsen, H. O. and Z. P. Bazant, "Uncertainty Analysis of Creep and Shrinkage Effects in Concrete Structures," Journal American Concrete Institute, Vol. 80, No. 2, March/April 1983, pp. 116-127.

36. Myers, B. L. and F. O. Slate, "Creep and Creep Recovery of Plain Concrete as Influenced by Moisture Conditions and Associated Variables," Magazine of Concrete Research, Vol. 22, No. 70, March 1970, pp. 37-41.

37. Nasser, K. W. and A. M. Neville, "Creep of Concrete at Elevated Temperatures," Journal American Concrete Institute, No. 62, December 1965, pp. 1567-79.

38. Ngab, A. S., A. H. Nilson, and F. O. Slate, "Shrinkage and Creep of High Strength Concrete," Journal American Concrete Institute, Title No. 78-22, July-August 1981, pp. 255-261.

39. Neville, A. M., Creep of Concrete: Plain, Reinforced, and Prestressed, North-Holland Publishing Co., Amsterdam, 1970.

40. Neville, A. M., Properties of Concrete, 3rd edition, Pitman Publishing, London, 1981.

41. Neville, A. M. and J. J. Brooks, "Time-Dependent Behavior of Concrete Containing a Plasticizer," Concrete, Vol. 9:10, October 1975, pp. 33-35.

42. Park, S. B. and M. Tia, "A Study of the Effects of Superplasticizer on the Engineering Properties of Plain Concrete," Unpublished submitted to Transportation Research Record, 1985.

43. Parrott, L. J., "An Examination of the Effects of Age at Loading Upon the Creep of Hardened Cement Paste," Magazine of Concrete Research, Vol. 25, No. 85, December 1973, pp. 197-200.

44. Parrott, L. J., "Increase in Creep of Hardened Cement Paste Due to Carbonation Under Load," Magazine of Concrete Research, Vol. 27, No. 92, September 1975, pp. 179-181.

45. SAS Users Guide, SAS Institute Inc., Cary, North Carolina, 1979.

46. Timusk, J. and R. S. Ghosh, "Maturing Creep of Portland Cement Paste," ACI Journal, Proceedings, Vol. 68, No. 12, December 1971.

47. Tokuda, H., M. Shoya, M. Kawakami, and N. Kagaya, "Application of Superplasticizer to Reduce Dry Shrinkage and Thermal Cracking of Concrete," Developments in the Use of Superplasticizers, ACI SP-68, American Concrete Institute, Detroit, 1981.

48. Tomosawa, F. and I. Fukushi, "Flowing (Superplasticized) Concrete," JCI Journal, Vol. 18, No. 7, July 1970, pp. 1-14.

49. Troxell, G. E., H. E. Davis, and J. W. Kelly, Composition and Properties of Concrete, 2nd Edition, McGraw-Hill, New York, 1968.

50. Walpole, Ronald E. and Raymond H. Myers, Probability and Statistics for Engineers and Scientists, 2nd Edition, Macmillan Publishing Co., New York, 1978.

51. Ward, M. A., "Creep of Air-Entrained Concrete," Magazine of Concrete Research, Vol. 21, No. 69, December 1969.

BIOGRAPHICAL SKETCH

Joseph M. Richardson was born November 29, 1944, in Louisville, Kentucky. His early elementary years were spent in Kingman, Arizona, where his father worked on Davis Dam. His family returned to its home state of Kentucky, and he graduated from Pleasure Ridge Park High School, Louisville, Kentucky, in 1962. He was graduated from Murray State University, Murray, Kentucky, in 1966, with majors in physics and History and a minor in mathematics. In 1969 he joined the U.S. Coast Guard where he was first operations officer aboard the buoy tender USCGC Hollyhock and then was a Rescue Coordination Center Controller. After discharge with a rank of Lt. j.g., he attended the University of Nevada--Reno, Nevada, and obtained a B.S.C.E. in 1974. He next worked for Phillips Petroleum Company as a construction engineer. In 1976 he was sent to Jakarta, Indonesia, as a construction superintendent. In 1978 he went to work for the Arabian American Oil Co. (ARAMCO) as first a project engineer in Dubai, United Arab Emirates, and later as a contract representative in The Netherlands. In 1981 he decided to obtain his master's degree in civil engineering and entered the University of Florida. He obtained a Master of Engineering degree in 1983. He is now working for his Ph.D., specializing in structural engineering.

Mr. Richardson was married in 1973 and has two children, age 7 and 8. He obtained his P.E. in 1979, and is a member of the National Society of Professional Engineers and the American Society of Civil Engineers.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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Byron E. Ruth, Chairman
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